INVESTIGATION OF THE LOWER BOUND COMPRESSIVE AND FLEXURAL STRENGTHS OF CONVENTIONAL CONCRETE AND CLAY MASONRY

by

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To my wife Deborah and my children Sophie, Audrey and David.

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ABSTRACT

INVESTIGATION OF THE LOWER BOUND COMPRESSIVE AND FLEXURAL STRENGTHS OF CONVENTIONAL CONCRETE AND CLAY MASONRY

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Conventional masonry is a handcrafted product in which units are placed by hand in a mortar bedding that is prepared during construction. TMS 402 *Building Code Requirements for Masonry Structures* (also referred to by the industry as the Masonry Standards Joint Committee Standard or MSJC Standard) and referenced standards consequently permit a range of various field conditions so as to allow practical construction (1).

The primary objective of the testing program was to measure a statistically significant number of test results to investigate the lower bound compressive and flexural strengths of conventional concrete and clay masonry under the permitted range of simulated field conditions.

96 compression tests and 144 flexural tests were performed on unreinforced, ungrouted masonry prisms. Half of these prisms consisted of hollow concrete masonry units (CMU) that were nominally 8 in. wide, 8 in. tall, and 16 in. long. The other half consisted of hollow brick units with standard modular brick dimensions.

Masonry prisms were constructed for this research with both Type S and Type N Portland Cement-Lime Mortars and both Type S and Type N Masonry Cement Mortars with the maximum permitted sand to cement ratio and the maximum permitted lime to cement ratio where applicable.

This research investigated the combined effects of constructing masonry with the maximum permitted mortar age, maximum permitted low and high curing temperatures which do not require special measures, minimum and maximum conventional initial rates of absorption (IRA), minimum and maximum conventional mortar water contents as determined by professional masons, and minimum and maximum conventional unit water contents.

For each of the 32 variations of mortar, the following mortar properties were recorded: mortar cube compressive strength, mortar flow, cone penetrometer resistance, and air content.

Half of each set of prisms was constructed with an "A" series of materials; and the other half was constructed with a "B" series of materials. All materials in the two series were mutually exclusive, obtained from different sources; and, two different professional masons were each assigned to a different series. The "B" sand was typical of common masonry sand used in North Texas and many other areas of the United States, having a much smaller average particle size and not complying with the gradation requirements of ASTM C 144 Sections 4.1 and 4.2.

The primary relevance of this investigation with regard to compressive strengths is to generate data for simulated field conditions that are unfavorable but currently permitted, to assist the design community in evaluating the common expectation among designers that field mortar cube compression strengths exceed the values of ASTM C 270 Table 2, though this practice is currently prohibited (2, 3, 4).

The primary relevance of this investigation with regard to flexural strengths is to assist TMS 402 in evaluating current permitted design strength assumptions considering current limitations on construction conditions. These provisions are the product of much debate and research over the last century. Historically, research has primarily been performed under standardized laboratory conditions even though there is wide recognition that the flexural strength of unreinforced, ungrouted masonry is a highly variable property. Failures of unreinforced, ungrouted masonry structures have been observed in the field during extreme load events and these provisions warrant further investigation (5, 6, 7, 8).

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CHAPTER 1

INTRODUCTION

This research investigated the lower bound compressive and flexural strengths of conventional concrete and clay masonry under simulated field conditions. In this chapter, general background information is first provided as a basic orientation for those who are not familiar with the masonry industry. An introduction of the research objectives is then provided with commentary on the relevance of this research, followed by a summary of currently permitted design assumptions for reference in the following chapters when the test program is described and the test results are interpreted.

1.1 Background

1.1.1 Conventional Concrete and Clay Masonry

Masonry, consisting of units bedded in mortar, has been used in construction around the world for millennia. What is considered conventional masonry has changed with time and still varies significantly between countries and even between regions within a country. Currently, concrete and clay masonry are both very common masonry systems used in new construction across the United States of America.

It is very common for concrete masonry construction to be either reinforced or unreinforced and to be designed to provide stability to the building under imposed loads as part of the load-bearing system and the in-plane lateral force resisting system as well as the primary exterior wall system that transfers out-of-plane loads to horizontal diaphragms and foundations or directly to shear walls. Reinforced concrete masonry with deformed bar reinforcement is typically either partially or fully grouted. Reinforced concrete masonry with bedjoint wire reinforcement is typically either fully grouted, partially grouted, or not grouted. Unreinforced concrete masonry is typically fully grouted or not grouted. One of the most common widths of concrete masonry walls is nominally 8 inches, with a specified width of 7 5/8 inches.

It is very common for clay masonry construction to be an unreinforced veneer system that is anchored to a structural system such as concrete masonry or stud construction. In recent years, some major manufacturing plants across the country have increased the area of holes in their primary production units to conform with ASTMC C 652 *Standard Specification for Hollow Brick* rather than ASTM C 62 *Standard Specification for Building Brick (Solid Masonry Units Made From Clay or Shale)* (9, 10). There are many different sizes of brick units commercially available today; two of the most common are king size brick and standard modular brick. Because the width of a king size brick is smaller while the face area is larger, king size brick is widely regarded as less expensive to produce, ship, and erect. Generally speaking, king size brick dominates the residential market while both king size and standard modular brick are used for commercial structures. For this research, standard modular units were used. The width of a standard modular brick wall is nominally 4 inches, with a specified width of 3 5/8 inches.

Generally speaking, the mortar joint is referred to as a bed joint when it horizontal and a head joint when it is vertical. And, the geometric bond pattern of masonry refers to the alignment of units from one course, which is a horizontal row of units, to the courses immediately above and below. (The flexural bond strength of masonry is different because it refers to the flexural resistance provided by a masonry assemblage at the mortar to unit interface when the assemblage is bent out of plane.) The term running bond (a geometric bond pattern) is used when the head joint of one course is located at the midpoint of the units above and below. The term stack bond (a geometric bond pattern) is used when all the head joints form a vertical line. Running bond is more common in new construction today.

1.1.2 Conventional Building Code References

In the United States of America, one of the most common building codes legally adopted by states and municipalities is the International Building Code (11). The 2009 International Building Code is the latest published version of this document and for masonry design it references TMS 402-08/ACI 530-08/ASCE 5-08 *Building Code Requirements for Masonry Structures*, which will be referred to as TMS 402 in this thesis. TMS 402 is also referred to by the masonry industry as the Masonry Standards Joint Committee Code, or MSJC Code. The first edition of the MSJC was published in 1988 by ACI and ASCE, before it was actually referred to as the MSJC and before TMS became the third sponsor organization for the document. TMS 402 references TMS 602-08/ACI 530.1-08/ASCE 6-08 *Specification for Masonry Structures*, which will be referred to as TMS 602 in this thesis rather than MSJC Specification (12). Both TMS 402 and TMS 602 reference numerous standards published by the American Society of Testing and Materials, or ASTM, which will be mentioned in this thesis, such as ASTM C 270 *Standard Specification for Mortar for Unit Masonry*.

1.1.3 Conventional Mortar Specification

ASTM C 270 permits three different categories of cement to be used in the construction of mortar: Portland Cement, Masonry Cement, and Mortar Cement. Both Portland Cement-Lime Mortar and Masonry Cement Mortar are commonly used across the United States and were used in this research; Mortar Cement Mortar is relatively new and not widely used. ASTM C 270 also specifies characteristics for 4 different types of mortar with each successively having a smaller percentage of cement than the previous one: M, S, N and O. Because mortar is generally less brittle when there is less cement, the industry recommendation is to use the weakest compressive strength mortar that satisfies the strength requirement needed (13). Type S and N are by far the most common types of mortar encountered in new construction above grade. Both Type S and N mortars are commonly specified for both concrete and clay masonry and both were used in this research. Type S Portland Cement-Lime Mortar, Type N Portland

Cement-Lime Mortar, Type S Masonry Cement Mortar, and Type N Masonry Cement Mortar were therefore the 4 mortar categories used to represent conventional masonry construction.

ASTM C 270 permits specification of mortar by two methods: the Proportion Method and the Property Method. Figure 1.1 shows a scan of ASTM C 270 Table 1, used in the Proportion Method, and ASTM C 270 Table 2, used in the Property Method. The default method is the Proportion Method, which is more commonly used. The Proportion Method provides acceptable ranges of relative volumes of cementitious materials and sand for a given mortar type. The Property Method provides acceptable ranges of properties (compressive strength, air content, water retention, ratio of sand to cementitious materials) for mortar prepared according to a proposed mortar batch design, providing relative volumes of cementitious materials and sand, at a standardized mortar water content range as evaluated by a standardized flow test. It is important to note that the purpose of both the proportion and property methods is to establish a mortar mix design to be used in batching relative volumes of cementitious materials and sand on a project. Mortar water content of field mortar is not specified because masons must account for many variables in the field such as evaporation and production rate to obtain mortar that is good considering various properties such as bond strength, shrinkage, compressive strength, durability, etc… Accounting for these variables requires a great deal of experience and most conventional masonry crews consist of multiple workers with varied levels of experience; therefore, errors are sometimes made when mixing and the more experienced masons must try to provide quality control before the mortar is placed, which can be difficult, especially given the recent declines in the number of experienced masons in many areas of the country.

The compressive strengths of mortar cubes made on job sites per ASTM C 780 are often compared with the minimum compressive strengths for mortar cubes listed in the ASTM C 270 Property Method as pass/fail criteria on projects (1800 psi for Type S mortar and 750 psi for Type N mortar). For several reasons, the masonry industry does not recommend this practice and has written several industry standard documents, including ASTM C 1586, so that this practice is prohibited (14). One reason is that masons must constantly adjust the mortar water content of newly batched mortar as field temperature and wind conditions vary so that the mortar water content when placed on the unit is compatible with the absorption rate of the unit to achieve a good bond between the materials. If this bond is poor the two materials can

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TABLE 1 Proportion Specification Requirements

NOTE—Two air-entraining materials shall not be combined in mortar.

Mortar	Type	Proportions by Volume (Cementitious Materials)								Aggregate Ratio (Measured in Damp, Loose Conditions)
		Portland Cement or Blended Cement	Mortar Cement			Masonry Cement			Hydrated Lime or Lime Putty	
			M	S	N	M	S	N		
Cement-Lime	M	1	...	...	...	...	...	...	¼	
	S	1	...	...	...	...	...	...	over ¼ to ½	
	N	1	...	...	...	...	...	...	over ½ to 1¼	
	O	1	...	...	...	...	...	...	over 1¼ to 2½	
Mortar Cement	M	1	...	...	1	...	...	...	...	Not less than 2¼ and not more than 3 times the sum of the separate volumes of cementitious materials
	M	...	1	...	...	...	...	...	...	
	S	½	...	...	1	...	...	...	...	
	S	...	...	1	...	...	...	...	...	
	N	...	...	...	1	...	...	...	...	
	O	...	...	...	1	...	...	...	...	
Masonry Cement	M	1	...	...	...	...	...	1	...	
	M	...	...	...	...	1	...	...	...	
	S	½	...	...	...	...	...	1	...	
	S	...	...	...	...	...	1	...	...	
	N	...	...	...	...	...	...	1	...	
	O	...	...	...	...	...	...	1	...	

TABLE 2 Property Specification Requirements[A]

Mortar	Type	Average Compressive Strength at 28 days, min, psi (MPa)	Water Retention, min, %	Air Content, max, %[B]	Aggregate Ratio (Measured in Damp, Loose Conditions)
Cement-Lime	M	2500 (17.2)	75	12	
	S	1800 (12.4)	75	12	
	N	750 (5.2)	75	14[C]	
	O	350 (2.4)	75	14[C]	
Mortar Cement	M	2500 (17.2)	75	12	Not less than 2 ¼ and not more than 3 ½ the sum of the separate volumes of cementitious materials
	S	1800 (12.4)	75	12	
	N	750 (5.2)	75	14[C]	
	O	350 (2.4)	75	14[C]	
Masonry Cement	M	2500 (17.2)	75	18	
	S	1800 (12.4)	75	18	
	N	750 (5.2)	75	20[D]	
	O	350 (2.4)	75	20[D]	

[A] Laboratory prepared mortar only (see Note 4).
[B] See Note 5.
[C] When structural reinforcement is incorporated in cement-lime or mortar cement mortar, the maximum air content shall be 12 %.
[D] When structural reinforcement is incorporated in masonry cement mortar, the maximum air content shall be 18 %.

Figure 1.1 Scan of ASTM C 270 Tables 1 and 2 separate such that there can be a structural concern for masonry that is unreinforced and ungrouted, a waterproofing concern for masonry that is on exterior walls, and an aesthetic concern for masonry that is exposed to view. Directing a mason to ensure mortar meets a minimum field cube compressive strength distracts the mason from the important task of constructing good integrated masonry. If there is a concern about compressive strength, this may nonetheless be valid. However, another reason the practice is prohibited is that mortar cubes are a very conservative test of in-place mortar conditions because the height-to-width ratio of a cube is much greater than for bedjoints. Furthermore, if the mortar in the cubes is taken from the mortar board, the water-cement ratio is greater than in-place conditions in which the units absorb some of the water, making the field cube test even more conservative. In addition, the values for mortar cube compressive strength required by the Property Method of ASTM C 270 were established to conservatively prove under laboratory conditions that a mortar batch design would not be a structural concern; these values were not established as a minimum standard for field cube strengths. And, finally, the curing conditions for ASTM C 780 mortar cubes are not the same as that in the wall. Therefore, ASTM C 270 does not permit designers to require that field mortar cubes exceed this high standard in the face of the double conservatism described above considering the fact that masons need to adjust the mortar water content of the mortar to create good, integrated masonry. If the mortar strength is a concern on a particular project, then the entire masonry construction on a project could be a concern because mortar is typically made and distributed throughout a project on all of the walls being constructed at that time. When 28 day field mortar cube strengths are below the Property Table values, legal arguments sometimes ensue with the engineer concerned about the safety of the product and the mason concerned about the cost of demolition and reconstruction he or she thinks unnecessary because industry standard documents state the product is acceptable. Recent research indicates that masonry prism compressive strengths for improperly batched mortar that is not permitted by current TMS 402 provisions could be slightly lower than the assumed design strengths that TMS 402 permits (15). That research was conducted in part to investigate the lower bound compressive strength of conventional masonry so as to better evaluate the industry recommended practice of not specifying that field mortar cube compression strengths meet a pass/fall criteria.

The maximum permitted sand to cement ratio by the Proportion Method of ASTM C 270 is 3:1. The maximum permitted lime to cement ratio in the Proportion Method is 1/2 to 1 for Type S mortar and 1 1/4 to 1 for Type N mortar.

ASTM C 270 requires that mortar sand conform with ASTM C 144 Sections 4.1 and 4.2, which have specific gradation requirements that are shown in Figure 1.2 (16).

4. Grading

4.1 Aggregate for use in masonry mortar shall be graded within the following limits, depending upon whether natural sand or manufactured sand is to be used:

Sieve Size		Percent Passing Natural Sand	Percent Passing Manufactured Sand
4.75-mm	(No. 4)	100	100
2.36-mm	(No. 8)	95 to 100	95 to 100
1.18-mm	(No. 16)	70 to 100	70 to 100
600-μm	(No. 30)	40 to 75	40 to 75
300-μm	(No. 50)	10 to 35	20 to 40
150-μm	(No. 100)	2 to 15	10 to 25
75-μm	(No. 200)	0 to 5	0 to 10

4.2 The aggregate shall not have more than 50 % retained between any two consecutive sieves of those listed in 4.1 nor more than 25 % between 300-μm (No. 50) and the 150-μm (No. 100) sieve.

Figure 1.2 Scan of ASTM C 144 Sections 4.1 and 4.2

However, it is common practice for commercial masons to use sands which do not comply with these gradation requirements because many natural sand deposits have finer gradations. ASTM C 144 Section 4.4 permits the use of such sands as long as the laboratory mortar can be prepared to comply with the Property Method requirements of ASTM C 270. To acknowledge this disconnect between what is commonly encountered and permitted in conventional practice versus ASTM C 144 Sections 4.1 and 4.2, half of the specimens for this research were prepared with sand having a gradation complying with ASTM C 144 Sections 4.1 and 4.2 and the other half were prepared with a common masonry sand, also known as brick sand, that does not comply with the gradation requirements of ASTM C 144 Sections 4.1 and 4.2 but was evaluated for compliance with ASTM C 144 Section 4.4 by Property Method tests.

1.1.4 Specified Compressive Strength of Masonry, f'_m

Compressive strengths can be critical for both reinforced and unreinforced masonry design. TMS 602 requires that the design professional specify f'_m, which is the specified compressive strength of masonry. Compression on masonry assemblages is resisted by portions of masonry units and mortar and often grout as well, which all have different properties. For structural analysis, TMS 402 permits designers to assume that portions of masonry units and mortar and grout all have the compressive strength of f'_m, which can then be considered analogous to f'_c for concrete design according to ACI 318 (17).

TMS 602 permits two different methods for determining f'_m: the Unit Strength Method and the Prism Method. The Unit Strength Method is more common because it permits determination of f'_m based only on the unit material (concrete or clay), unit (compressive) strength, and mortar type (M,S,N). The Prism Method is based on standardized prisms constructed and tested per ASTM C 1314, which can be problematic (18). One problem is that many construction materials testing laboratories do not have the capability of testing full-scale concrete prisms properly because ASTM C 1314 requires a very thick steel platen which is expensive and often does not fit inside of many concrete cylinder compression machines. Therefore prism compression testing can be expensive. And, perhaps more importantly, prism testing can require redesign if it is discovered after field prisms are tested at 28 days that an assumed f'_m value appears unconservative. The prism results in this thesis are therefore compared with the values permitted by the Unit Strength Method.

TMS 602 Tables 1 and 2, used in the Unit Strength Method for clay and concrete masonry respectively, are shown in Figure 1.3.

Table 1 — Compressive strength of masonry based on the compressive strength of clay masonry units and type of mortar used in construction

Net area compressive strength of clay masonry units, psi (MPa)		Net area compressive strength of masonry, psi (MPa)
Type M or S mortar	Type N mortar	
1,700 (11.72)	2,100 (14.48)	1,000 (6.90)
3,350 (23.10)	4,150 (28.61)	1,500 (10.34)
4,950 (34.13)	6,200 (42.75)	2,000 (13.79)
6,600 (45.51)	8,250 (56.88)	2,500 (17.24)
8,250 (56.88)	10,300 (71.02)	3,000 (20.69)
9,900 (68.26)	—	3,500 (24.13)
11,500 (79.29)	—	4000 (27.58)

Table 2 — Compressive strength of masonry based on the compressive strength of concrete masonry units and type of mortar used in construction

Net area compressive strength of concrete masonry units, psi (MPa)		Net area compressive strength of masonry, psi[1] (MPa)
Type M or S mortar	Type N mortar	
—	1,900 (13.10)	1,350 (9.31)
1,900 (13.10)	2,150 (14.82)	1,500 (10.34)
2,800 (19.31)	3,050 (21.03)	2,000 (13.79)
3,750 (25.86)	4,050 (27.92)	2,500 (17.24)
4,800 (33.10)	5,250 (36.20)	3,000 (20.69)

[1] For units of less than 4 in. (102 mm) height, 85 percent of the values listed.

Figure 1.3 Scan of TMS 602-08 Tables 1 and 2, for the Unit Strength Method

1.1.5 Modulus of Rupture, f_r

Flexural bond strengths are often critical for the design of unreinforced masonry structures, and can be critical for veneers if they are designed with supports that are far apart.

As shown in Figure 1.4, Chapter 3 of TMS 402 provides values for the modulus of rupture, f_r, which indicate the anticipated flexural tensile strength for various configurations of units, grouting, bond (e.g. stack bond or running bond) and direction of loading. It is important to note that these values are independent of the type of masonry unit (e.g. clay or concrete). These values are used in Strength Design.

Table 3.1.8.2 — Modulus of rupture, f_r, psi (kPa)

Direction of flexural tensile stress and masonry type	Mortar types			
	Portland cement/lime or mortar cement		Masonry cement or air entrained portland cement/lime	
	M or S	N	M or S	N
Normal to bed joints in running or stack bond				
Solid units	100 (689)	75 (517)	60 (413)	38 (262)
Hollow units[1]				
Ungrouted	63 (431)	48 (331)	38 (262)	23 (158)
Fully grouted	163 (1124)	158 (1089)	153 (1055)	145 (1000)
Parallel to bed joints in running bond				
Solid units	200 (1379)	150 (1033)	120 (827)	75 (517)
Hollow units				
Ungrouted and partially grouted	125 (862)	95 (655)	75 (517)	48 (331)
Fully grouted	200 (1379)	150 (1033)	120 (827)	75 (517)
Parallel to bed joints in stack bond				
Continuous grout section parallel to bed joints	250 (1734)	250 (1734)	250 (1734)	250 (1734)
Other	0 (0)	0 (0)	0 (0)	0 (0)

[1] For partially grouted masonry, modulus of rupture values shall be determined on the basis of linear interpolation between fully grouted hollow units and ungrouted hollow units based on amount (percentage) of grouting.

Figure 1.4 Scan of TMS 402-08 Table 3.1.8.2 providing Modulus of Rupture Values

The allowable flexural tensile stresses for clay and concrete masonry, from Chapter 2 of TMS 402, are shown in Figure 1.5 and used in Allowable Stress Design. The modulus of rupture values used for Strength Design were established by multiplying the Allowable Stress Design values by a factor of 2.5, which was consistent with other conversions of allowable values to nominal strengths at the time the conversion was made.

Table 2.2.3.2 — Allowable flexural tensile stresses for clay and concrete masonry, psi (kPa)

Direction of flexural tensile stress and masonry type	Mortar types			
	Portland cement/lime or mortar cement		Masonry cement or air entrained portland cement/lime	
	M or S	N	M or S	N
Normal to bed joints				
Solid units	40 (276)	30 (207)	24 (166)	15 (103)
Hollow units[1]				
Ungrouted	25 (172)	19 (131)	15 (103)	9 (62)
Fully grouted	65 (448)	63 (434)	61 (420)	58 (400)
Parallel to bed joints in running bond				
Solid units	80 (552)	60 (414)	48 (331)	30 (207)
Hollow units				
Ungrouted and partially grouted	50 (345)	38 (262)	30 (207)	19 (131)
Fully grouted	80 (552)	60 (414)	48 (331)	30 (207)
Parallel to bed joints in stack bond				
Continuous grout section parallel to bed joints	100 (690)	100 (690)	100 (690)	100 (690)
Other	0 (0)	0 (0)	0 (0)	0 (0)

[1] For partially grouted masonry, allowable stresses shall be determined on the basis of linear interpolation between fully grouted hollow units and ungrouted hollow units based on amount (percentage) of grouting.

Figure 1.5 Scan of TMS 402-08 Table 2.2.3.2 providing Allowable Flexural Tension Values

Section 2.2.3.2 of the Commentary to TMS 402 states, “Allowable flexural tensile stresses for portland-cement lime are traditional values.” and, later, “For masonry cement and air entrained portland-cement lime mortar, there are no conclusive research data and, hence, flexural tensile stresses are based on existing requirements in other codes.” And, also, “Variables affecting tensile bond strength of brick masonry normal to bed joints include mortar properties, unit initial rate of absorption, surface condition, workmanship, and curing condition.”

Failures of unreinforced, ungrouted concrete and clay masonry structures have been observed after extreme load events such as hurricanes, tornadoes, and earthquakes (5, 6, 7, 8). It is not certain to what extent the flexural bond strength of such masonry may have failed to perform in some of these failures or whether the load was higher than current industry construction standards such as the IBC, ASCE 7 and TMS 402 have established to be minimum design loads. However, the mode of failure in some of these structures has been identified as flexural failure.

1 .1.6 Design Methods Permitted by TMS 402

TMS 402 permits three distinctly different methods of designing masonry for various applications: Allowable Stress Design (ASD), Strength Design (SD), and Empirical Design (Empirical). The methods sometime yield different results and TMS 402 has been addressing many of these differences with each new publication.

Empirical Design only applies to unreinforced masonry and is based on methods used historically, providing maximum permitted gross area compressive stresses for dead and live loads only and providing maximum permitted length to thickness ratios shown in Figure 1.6. TMS 402 only permits Empirical under very limited conditions and it is not certain how many designers continue to use Empirical today.

Table 5.5.1 — Wall lateral support requirements

Construction	Maximum *l/t* or *h/t*
Bearing walls	
Solid units or fully grouted	20
All other	18
Nonbearing walls	
Exterior	18
Interior	36

In computing the ratio for multiwythe walls, use the following thickness:

1. The nominal wall thicknesses for solid walls and for hollow walls bonded with masonry headers (Section 5.7.2).
2. The sum of the nominal thicknesses of the wythes for non-composite walls connected with wall ties (Section 5.7.3).

Figure 1.6 Scan of TMS 402-08 Table 5.5.1 Empirical Height-to-Thickness Limits

ASD was developed as a rational method of limiting stresses to allowable values. These allowable values were developed over the last century starting with historical data, with research continually unfolding, and these values have been modified somewhat with time based on changes to industry standards. TMS 402-08 permits increasing these allowable stresses by one-third when considering load combinations with wind and/or seismic loads. Because these are the primary cases that govern most designs, this thesis will compare test results in this study to the current allowable stresses increased by one-third. The TMS 402 – committee-approved-working-draft of revisions to TMS 402-08 dated November 15, 2009 incorporates a change whereby allowable stresses for unreinforced, ungrouted masonry in flexure have been increased by one-third and the provision allowing designers to increase the stresses by one-third for cases with wind or earthquake has been removed (19). Therefore, this change could take effect in the next edition of TMS 402. Allowable compressive stresses are established by multiplying a fraction times f'_m, with the fraction varying depending on how much of the compression is due to axial load and how much is due to bending, as well as slenderness effects. However, allowable flexural tension is provided in a table with no indication of what the expected value should be. As with most material standards that use an allowable stress design approach, the factor of safety can be established by the ratio of the load to the strength, accounting for variability in both loads and resistance characteristics simultaneously based on the judgment of the industry several decades ago before SD was developed.

SD was developed for masonry within the last few decades. The maximum permitted design strengths are determined based on permitted strength assumptions and a corresponding strength-reduction factor. The load factors common to ASCE 7 and the IBC for strength design load combinations are to be used with TMS 402 strength design provisions. According to TMS 402 Commentary, the permitted SD strength assumptions were established by multiplying the ASD values by a factor of 2.5 to determine the nominal strength, which was consistent at the time with other values that were multiplied by the same factor when converting allowable values to nominal strengths. In other words, the modulus or rupture values and the strength-reduction factors were not established on an independent statistical understanding of the variability of flexural bond strength. While the TMS 402 – committee-approved-working-draft of revisions to TMS 402-08 dated November 15, 2009 incorporates a change in ASD that increases the allowable stresses by one-third, as shown in Figure 1.7 and removes the provision that permits designers to increase allowable stresses, there is no change that increases the modulus of rupture values for unreinforced, ungrouted behavior by one-third for SD.

Table 2.2.3.2 — Allowable flexural tensile stresses for clay and concrete masonry, psi (kPa)

Direction of flexural tensile stress and masonry type	Mortar types			
	Portland cement/lime or mortar cement		Masonry cement or air entrained portland cement/lime	
	M or S	N	M or S	N
Normal to bed joints				
Solid units	~~40 (276)~~53 (366)	~~30 (207)~~40 (276)	~~24 (166)~~32 (221)	~~15 (103)~~20 (138)
Hollow units[1]				
Ungrouted	~~25 (172)~~33 (228)	~~19 (131)~~25 (172)	~~15 (103)~~20 (138)	~~9 (62)~~12 (83)
Fully grouted	~~65 (448)~~86 (593)	~~63 (434)~~84 (579)	~~61 (420)~~81 (559)	~~58 (400)~~77 (531)
Parallel to bed joints in running bond				
Solid units	~~80 (552)~~106 (731)	~~60 (414)~~80 (552)	~~48 (331)~~64 (441)	~~30 (207)~~40 (276)
Hollow units				
Ungrouted and partially grouted	~~50 (345)~~66 (455)	~~38 (262)~~50 (345)	~~30 (207)~~40 (276)	~~19 (131)~~25 (172)
Fully grouted	~~80 (552)~~106 (731)	~~60 (414)~~80 (552)	~~48 (331)~~64 (441)	~~30 (207)~~ 40 (276)
Parallel to bed joints in ~~stack~~ masonry not laid in running bond				
Continuous grout section parallel to bed joints	~~100 (690)~~133 (917)	~~100 (690)~~133 (917)	~~100 (690)~~ 133 (917)	~~100 (690)~~ 133 (917)
Other	0 (0)	0 (0)	0 (0)	0 (0)

Figure 1.7 Revised Table 2.2.3.2 Allowable Flexural Tension Values for ASD from the TMS 402-Committee-Approved Working Draft of Revisions to TMS 402-08 dated November 15, 2009

1.1.7 Field Variations Permitted by TMS 402

Conventional masonry is a handcrafted product in which units are placed by hand in a mortar bedding that is prepared during construction. TMS 402 and referenced standards consequently permit a range of various field conditions so as to allow practical construction. Section 1.18 of TMS 402 provides minimum inspection requirements which generally involve visual field inspections and recording certifications. The design values permitted by TMS 402 are based on research and experience that generally accounts for variability of workmanship but not all important sources of variation that are currently permitted by TMS 402 during construction. Researchers have struggled with the variability of flexural strengths in unreinforced, ungrouted masonry over the last century, often noting that it is very difficult to account for the effects of all field variables that could decrease these strengths.

ASTM C 270 prohibits use of mortars after 2 1/2 hours after mixing. Prior to that time, the mortar is permitted to be retempered by adding water as frequently as needed to restore the required consistency. As with all cementitious materials, the hydration process as cement reacts with water forms fragile molecular bonds in the first few hours. If mortar is mixed periodically for the full permitted duration, it is therefore generally acknowledged that this could decrease the compressive and flexural strength of both the mortar and the masonry assemblage.

When the ambient temperature is below 40°F, TMS 602 requires special measures for cold weather construction such as heating water to at least 40°F. When the ambient temperature is above 90°F and the wind velocity is greater than 8 mph, TMS 602 requires special measures for hot weather construction such as fog spraying newly constructed masonry until damp at least three times a day until the masonry is three days old. Presumably TMS 602 would permit unlimited temperatures in a protected environment where the wind velocity does not exceed 8 mph.

The initial rate of absorption, IRA, is an indicator of how much water a brick unit will absorb when sitting on the surface of a shallow bed of water in 1 minute. A higher IRA value indicates the unit absorbs more water. Masons must be generally aware of the IRA of clay brick units. High or low IRA units with an incompatible mortar water content can result in poor flexural bond strength. If clay units have an IRA which exceeds 1 g/min/in.^{2}, section 3.2 C of TMS 602 requires that clay units be wetted so that the IRA no longer exceeds this value. In ASTM C 67, IRA values are reported in terms of g/min/30 in^2. One must divide IRA values as reported per ASTM C 67 by 30 to compare with TMS 602 provisions.

TMS 602 prohibits deliberate wetting of clay units having an IRA less than 0.2 g/min/in^2. And, TMS 602 requires that units be placed when surface dry, so that there is no visible standing water on the surface. However, TMS 602 does not otherwise limit the moisture in a unit. It is not uncommon for rain to increase the moisture contents of units in the field immediately prior to installation and certainly after installation during construction.

ASTM C 270 requires that all materials be mixed “with the maximum amount of water to produce a workable consistency”. The range of mortar water contents permitted therefore is subjectively based on what each mason regards to be a workable consistency, which will vary depending on the nature of the work. For example, when working in a relatively hot climate it is not uncommon for conventional masonry crews to increase the mortar water content even though some areas of construction may be shaded as the construction progresses. Furthermore, it is not uncommon for less experienced laborers to dose water too high or too low because it is sometimes difficult to gauge, leaving the more experienced masons laying units with a decision to spend time and money correcting the problem or to accept mortar that still has a workable consistency even though it may not be exactly the way they would have mixed it. This arrangement is an essential part of the learning experience that has always been and will always be a part of masonry construction. As an example with drier mortar, when replacing small sections of masonry after the original construction is completed (which can be necessary for a host of reasons such as if units are damaged during or after construction, if units are installed with the incorrect architectural finish, if other trades require access into the cells of the wall, and if there is a change order) it is not uncommon for conventional masonry crews to decrease the mortar water content so as to create a firm enough bedding to support the new masonry units which often cannot be laid in the normal manner, and to be able to push mortar into position above and around the new units.

1.2 Research Objectives And Relevance

The primary objective of the testing program was to perform a statistically significant number of tests to investigate the combined effect of the various simulated field conditions which are permitted by current TMS 402 provisions and described as the testing program in Chapter 3 on the compressive and flexural bond-wrench strengths of conventional concrete and clay masonry prisms that are unreinforced and ungrouted.

The primary relevance of this investigation with regard to compressive strengths is to assist the design community in evaluating whether or not it is necessary for safety to specify minimum field mortar cube compression strengths as a method of field quality control (despite that fact that industry standard documents prohibit this practice).

The primary relevance of this investigation with regard to flexural strengths is to assist TMS 402 in evaluating the historic use of allowable stress flexural tension values for unreinforced, ungrouted masonry. It is widely recognized that the flexural strength of unreinforced, ungrouted masonry is a highly variable property. In addition, flexural failures of unreinforced, ungrouted masonry structures and elements have been observed in the field (5, 6, 7, 8). Over the last century, research was performed as earlier versions of the current TMS 402 provisions were being developed. Currently, TMS 402 provides permitted design strength assumptions and provides TMS 602 specifications that limit many field conditions which effect flexural strength. There is no known research which evaluates flexural strengths of unreinforced, ungrouted masonry under the extremes of the limitations defined entirely by current TMS 402 and 602 provisions. The objective of this research with regard to flexural strength was therefore to measure the effect of the permitted field variations tested compared with the conditions which were typical of most research used to justify the current permitted strength assumptions in TMS 402.

The objective of this research was not to provide a statistical evaluation of the probability that similar conditions as those created in this research may occur in the field. However, some discussion is provided on the engineering judgment necessary to evaluate the research results.

To fulfill the primary objective, many different types of conventional masonry prisms were constructed. It was not the objective of this research to compare different types of construction to one another. In some limited cases, one may be able to draw some tentative theories regarding such comparisons. However, in most cases one should note that numerous conditions varied from one set of prisms to another which would blur such comparisons.

To fulfill the primary objective, numerous conditions were intentionally varied simultaneously during the preparation of many prisms. It was not the objective of this research to isolate the effects of one variable versus another. Furthermore, it was not the objective of this research to determine the absolute lowest compressive or flexural strengths for masonry.

CHAPTER 2

LITERATURE SURVEY

A literature search was conducted to better understand the historical development of the current TMS 402 design provisions and TMS 602 specifications.

2.1 TMS Disaster Investigations

A review of many reports created for the TMS Disaster Investigation Program identified numerous different failure modes in existing masonry structures and veneers under extreme loads such as hurricanes, tornadoes and earthquakes. Identification of failure modes is difficult during a disaster investigation and some modes are easier to identify than others. Many failures were attributed to poor connections, which is not relevant to this research.

No occasions were noted in which a failure was attributed to over-compression of unreinforced masonry. Signs of compression in masonry shear walls were observed such as by distortion or failure of cladding materials in a few instances. Some compressive failures of reinforced masonry in earthquakes were noted in some instances with poor grouting or over-reinforced cross-sections. However, both of these conditions are addressed by current TMS 402 and TMS 602 provisions regarding execution of grouting, inspection and maximum reinforcement ratios. Therefore, compression of masonry that complies with TMS 402 and TMS 602 does not appear to be a common failure mode; however, knowledge of the compressive strength of masonry is an important part of good masonry design as it relates to maximum reinforcement ratios and durability.

Flexural failures of unreinforced, ungrouted masonry, were clearly identified in many reports (5, 6, 7, 8). Common failures occurred in minor structures such as signs and fences, as well as cantilevered elements on buildings such as parapets or chimneys. However, occasions were also noted of structural building failures that can be attributed to exceeding the flexural capacity of unreinforced, ungrouted masonry. This implies that it is important to have confidence that the actual strength of unreinforced, ungrouted masonry meets or exceeds the assumed strength.

2.2 TMS 402 and TMS 602 Commentary and References

A review of current and previous editions of TMS 402 and TMS 602 Commentaries and References identified some of the historical rationale for provisions. ACI 530-88/ASCE 5-88 was the first edition of the document which is now TMS 402-08/ACI 530-08/ASCE 5-08 (20).

The Unit Strength Method values for concrete in ACI 530-88 are identical to those in TMS 402-08 (with an editorial exception for units having a unit strength less than the minimum 1900 psi required by ASTM C 90). The Unit Strength Method values for clay in ACI 530-88 were somewhat more conservative than for TMS 402-08. The three graphs indicating compressive strength data that are shown in the Commentary for ACI 530-88 are identical (with the exception of some editorial changes) to the current three graphs in TMS 402-08 Commentary. The references listed in ACI 530-88 in the discussion on the Unit Strength Method are dated between 1955 and 1980. The references listed in ACI 402-08 in the same discussion are dated between 1955 and 1999. The references chosen appears to indicate that the compressive values in ACI 530-88 were based primarily on research performed after World War II, with a consensus being established by 1988. Some research has been performed since 1988 which was the basis for an adjustment in the clay masonry values as further research was been performed. A review of these sources, however, indicates that most of all the research used as the basis for the Unit Strength Method was performed on average conditions, not extreme conditions which are often encountered in the field.

The allowable flexural tension values in ACI 530-88 for unreinforced, ungrouted concrete masonry normal to bed joints in running bond for portland cement/lime mortars are identical to the allowable flexural tension values in TMS 402-08. It appears that the values for masonry cement mortars were originally based primarily on a historical percentage reduction from the values for portland cement/lime mortars. In the years immediately after ACI 530-88 was published, research was performed and acknowledged by reducing the values for masonry cement mortars from what they were in ACI 530-88. The values for clay masonry were originally lower than those for concrete; however, the clay values were changed to match the current concrete values in a subsequent edition during the 1990's. The references listed in the ACI 530-88 Commentary discussion on flexural tensile strength of unreinforced, ungrouted masonry are dated between 1982 and 1985. The references listed in the TMS 402-08 Commentary for the same discussion are dated between 1982 and 1996. The discussions clearly indicate that flexural tension is a highly variable property that is influenced by many different parameters.

2.3 Other Sources

Many other resources were reviewed, including TMS Journal papers, papers from the North American Masonry Conference and other conferences, research from Universities including dissertation and thesis papers, as well as educational materials including textbooks, materials from masonry organizations, and magazine articles.

2.3.1 Resources on the Compressive Strengths of Reinforced and Unreinforced Masonry

It is clear from magazine articles that there is a great confusion regarding the proper application of field mortar cube strengths for quality control purposes because the masonry industry in the United States does not recommend or require a minimum field cube strength while members of the design and testing community at large often believe that a minimum should be necessary just like for concrete cylinders (2). There has been interest in creating a performance requirement, a minimum field cube compressive strength specification for purposes of quality control, including some work that has been performed in Canada (21, 22). Many people in the design community refer to ASTM C 270 Table 2 values out of ignorance, caution, or defiance, even though these values are not intended for quality assurance. The

Property Specification of the Canadian Standard, CSA A179, requires 28 day job prepared mortar cube strengths be approximately two-thirds the values in ASTM C 270 Table 2. (23)

Some research has been performed since the values in the Unit Strength Method were last modified (24). However, most other research concerning the compressive strength of masonry has explored the statistical distribution of data, accounting for general workmanship variability under relatively standardized conditions. A significant amount of research has explored the influence of factors such as outside curing versus laboratory curing, and some research has explored the influence of temperature ranges. Data across the nation is available concerning the statistical variation of field-constructed concrete masonry prisms (25). In addition, research shows that extending multiple field conditions well beyond the limits permitted by TMS 402-08 and TMS 602-08 could cause a reduction of the compressive strength of masonry prisms to below the Unit Strength Method values (26). However, no compressive strength research was identified that tests the combined influence of extreme field conditions within the ranges permitted by the current provisions of TMS 402-08 and TMS 602-08. Therefore, there is a need for research investigating the lower bound compressive strengths for conventional concrete and clay masonry under these permitted field conditions.

2.3.2 Resources on the Flexural Strengths of Unreinforced, Ungrouted Masonry

Historical documents, including England's building codes from past centuries and American design handbooks from the early 1900's were found to have many limiting conditions which are simply not included in the current Empirical provisions of TMS 402 (27, 28).

Five binders of resources from the 1900's concerning the flexural tensile strength of masonry, collected by Clayford T. Grimm of the University of Texas at Austin before passing away (currently in the possession of Dr. Matthys of the University of Texas at Arlington), were also reviewed as part of this literature search. This collection was an invaluable resource.

There is a significant amount of research on the flexural tensile capacity of unreinforced, ungrouted masonry. More data is available for bond wrench tests than for full scale or smaller scale wall panels. Much small specimen testing was done earlier with a thirdpoint loading procedure rather than the modern bond wrench test. In addition, much of the earlier data is presented with regard to gross areas rather than net areas, especially data for clay brick masonry which until recently has been presented mostly in terms of gross area. Significantly more research has been done with clay brick and concrete brick prisms rather than larger units. In addition, some of the literature suggests that only full-scale wall or smaller scale wall panel testing should be used to evaluate flexural bond, recommending that bond wrench tests only be used for comparative purposes. Many have questioned the suitability of Empirical.

"A Matrix Compilation of Masonry Flexural Bond Test Data" for the MSJC Task Group on Tensile Bond dated February 1991 is a resource that was developed using data from 34 different resources for the purposes of creating an objective matrix of data for side-by-side comparison (29). The resource states in the Introduction, "The document is also a resource to user's of the Code so that they are informed about the available data used in establishing allowable tensile bond values." It is clear from this resource that all of the field variations permitted by TMS 402 and TMS 602 in combination were not tested in any of the research tests. On the contrary, the research was generally standardized conditions.

The following quote characterizes the frustration researchers have had trying to define the undefinable range of possible flexural tensile strengths (30).

> "While certain trends have been identified, the large number and range of variables have to date made it impossible to develop any quantifiable set of guidelines or specifications to assure consistent levels of tensile bond between mortars and masonry units." "In design standards, the current approach is to assign specific allowable tensile bond stresses to combinations of mortars and masonry units which satsify the appropriate material standards. However, this practice must either result in very low values because of the range of results, or allow higher values which have been to some extent callibrated to full-scale tests."

In addition, the following quote from a 6 page summary of the state of the art published by the Portland Cement Association characterizes the sensitive nature of flexural bond, indicating that it cannot be accurately predicted without more information than TMS 402 currently requires to establish the permitted flexural tensile strength for design purposes (31). For convenience, this entire 6 page document is provided in Appendix A.

> "Development of bond strength in masonry is a complex process dependent on many variables related to materials, fabrication, curing and testing. Several of these parameters are interdependent. Certain general relationships have been established by isolating and measuring the effect of specific variables. However, it is important to note that relationships exhibited under controlled experimental conditions may be obscured in actual application by the effect of changes in other parameters. Perhaps the most significant finding that can be gleaned from a review of the numerous investigations with respect to bond strength is the observation that it is a combined property of the mortar and the unit together. It cannot be accurately predicted from individual characteristics of the component materials."

2.4 TMS 402-Committee Approved Working Draft Revisions to TMS 402-08 dated November 15, 2009

The TMS 402-Committee Approved Working Draft Revisions to TMS 402-08 dated November 15, 2009 incorporates a change to the permitted ASD design flexural tensile strengths for unreinforced, ungrouted masonry, but not SD values. These values for ASD have been increased by one-third as the document also incorporates a change which removes the former ASD provision that permits increasing stresses by one-third for load combinations with wind or earthquake. This change was initiated by an earlier change by ASCE 7 which revised load combinations for ASD and prohibited use of one-third stress increase provisions from material standards. ACI 530-88 Commentary stated the following regarding the one-third stress increase before ASCE 7 changed the ASD load combinations:

> "5.3.2 Previous editions of building codes have customarily used a higher allowable stress when considering wind or earthquake in a structure. This increase has come under attack from some quarters and there has been some confusion as to the rationale for permitting the increase. The committee recognizes the situation but has opted to continue to increase allowable stresses in the traditional manner until documentation is available to warrant a change."

The recent change to the one-third stress increase by TMS 402 was part of a larger revision that changed stresses for reinforcement and masonry in other areas of the document. This change was evaluated by the TMS 402 Committee with respect to statistical data available at the time. The values for unreinforced, ungrouted masonry were changed in part on the basis of a recent study which evaluated the reliability of masonry in flexure using a log normal statistical assumption to establish the 95% characteristic value (32). However, this study noted the limited availability of data for some conditions recommending further research, "However, the authors feel that looking at panel tests and bond wrench tests would be useful before adopting the tentative proposed changes."

These changes in the working draft will become part of the next edition of TMS 402 unless there is action otherwise.

2.5 Need for Research

Based on the above information, it is clear that there is a need for an investigation of lower bound compressive and flexural strengths for conventional concrete and clay masonry, exploring the combined effects of extreme field conditions within the ranges currently permitted by TMS 402 and TMS 602.

CHAPTER 3

TEST PROGRAM OVERVIEW

One of the easiest ways to keep track of the many different types of the primary specimens that were part of the test program is to understand the seven character specimen identification system that was used throughout the research, as illustrated in Figure 2.1.

C M S A W F 3

Unit Type

C = Concrete

B = Brick (Clay Brick)

Cement Type

P = Portland Cement-Lime

M = Masonry Cement

Mortar Type

S = Type S Mortar

N = Type N Mortar

Product Series

A = “A” Products

B = “B” Products

Mortar Water Content

C = Control

W = Wet Limit

D = Dry Limit

Specimen Type

C = Compression Prism

F = Flexural Prism

U = Mortar Cube

Specimen Number

1, 2 or 3

Figure 2.1 Illustration of the Seven Character Specimen Identification System

Throughout this document, when referencing specimens, the numbers on the right are not shown when it is not necessary to refer to a particular specimen. In addition, throughout this document, dashes are shown instead of the alphabetical characters when all possible alphabetical characters apply. For example, " - P S A D - " mortar refers to the mortar used for both concrete and clay brick masonry, for Portland Cement-Lime Mortar, Type S, using "A" Series Products, in dry limit conditions, for both compression and flexural prisms as well as mortar cubes.

3.1 Description and Distribution of Masonry Prisms

There were 240 masonry prisms constructed and tested at 28 days as the primary part of this research. As noted by the first character in Figure 2.1, half of these prisms were concrete masonry prisms and half were clay brick masonry prisms. As noted by the second character, half of each set was constructed with portland cement-lime mortars and half of each set was constructed with masonry cement mortars. As noted by the third character, half of each subsequent set was constructed with Type S mortar and the other half with Type N mortar. As noted by the fourth character, half of each remaining set was prepared with products from an "A" series of products and the other half from a "B" series of products. The 15 prisms in each remaining set can then be divided into five groups because, as indicated by the last character, each of the unique groups consisted ultimately of three prisms so that an average would be representative of the tested conditions. As indicated by the second-to-last character, two of these five groups were tested in compression and the remaining three were tested in flexure. Of the two compression groups, one was constructed with the control mortar water content established by the mason and the other was constructed with wet limit mortar water content acceptable to the mason with wet units that were sprayed with water several times in the days preceding construction to simulate rain. Of the three flexural groups: one was constructed with the control mortar described above; one was constructed with the wet limit mortar described above; and one was constructed with the dry limit mortar water content acceptable to the mason. The mortar water content under the “dry limit” condition complied with the ASTM C 270 Section 7.3 requirement “…with the maximum amount of water to produce a workable consistency” because the mason was directed to prepare mortar for repair or detailed work which required a stiff mortar. The only units sprayed with water to simulate rain were those constructed with wet limit mortar. The masons did not wet any units. The ages of the mortars at initial placement were approximately 15 minutes for the control mortars. The ages of the mortars at initial placement were approximately 2 hours for both the wet limit mortars and the dry limit mortars, which were both retempered only once. All specimens with control mortar were cured at approximately 75°F. All other specimens with “A” products were cured at approximately 40°F; with “B” products, 90°F. Both the concrete compression and flexural prisms were two units high. The clay brick compression prisms were three units high; the clay brick flexural prisms were four units high.

3.2 Mortar Tests

For each of the 32 variations of mortar described above, the following mortar properties were recorded: mortar cube compressive strength, mortar flow, cone penetrometer resistance, and air content. Fresh mortar properties were measured within approximately 5 to 10 minutes after initial mixing. For the wet limit and the dry limit mortars, the mortar flow and cone penetrometer resistance were again measured after retempering, which was completed approximately 2 hours after initial mixing.

For these tests, mortars were identified using the same seven character specimen identification system used for prisms; however, a dash was inserted where either all or no character options applied. For example, wet limit mortars were used on both concrete and brick prisms so both unit types applied and the first character for wet limit mortars is a dash.

3.3 “A” and “B” Product Series

All materials in the two series were mutually exclusive, obtained from different sources; and, two different professional masons were each assigned to a different series.

The “B” sand was typical of common masonry sand used in North Texas and many other areas of the United States, having a much smaller average particle size and not complying with the gradation requirements of ASTM C 144 Sections 4.1 and 4.2. There is an exception that permits such sands if the mortars proposed can be constructed to pass the series of tests required by the Property Method of ASTM C 270. Therefore those property tests were performed on the “B” series mortars under the standardized laboratory flow required by ASTM C 270.

More detail about the products and masons is provided in the following chapters.

3.4 Miscellaneous Other Tests

In addition to the above tests, several other tests were performed such as aggregate sieve analyses, net area measurements, unit compressive strength tests, initial rate of absorption measurements, and unit water content measurements of the units which were sprayed to simulate rain.

CHAPTER 4

PROCEDURES, MATERIALS AND EQUIPMENT

4.1 Procedures

Except where noted otherwise, the procedures described in this thesis were performed in accordance with the following ASTM standards. Where applicable, ASTM standards referenced by the following ASTM standards were also used:

ASTM C 67-07: Standard Test Methods for Sampling and Testing Brick and Structural Clay Tile.

ASTM C 140-06: Test Methods for Sampling and Testing Concrete Masonry Units and Related Units.

ASTM C 144-04: Specification for Aggregate for Masonry Mortar.

ASTM C 270-07: Standard Specification for Mortar for Unit Masonry.

ASTM C 780-06a: Test Method for Preconstruction and Construction Evaluation of Mortars for Plain and Reinforced Unit Masonry.

ASTM C 1072-06: Test Method for Measurement of Masonry Flexural Bond Strength

ASTM C 1314-09: Standard Test Method for Compressive Strength of Masonry Prisms.

ASTM C 1552-09a: Practice for Capping Concrete Masonry Units, Related Units and Masonry Prisms for Compression Testing

4.2 Materials

4.2.1 Cementitious Materials

The following is a description of each of the 8 cementitious materials used in this research. For each type of cementitious material, the “A” and “B” materials were from different sources. Some samples of each material were sent to a commercial laboratory and the specific gravities of the solid particles were measured as tabulated in Tables 4.1 and 4.2. The manufacturer's bags indicated the unit weights for each package shown in these Tables.

The "A" Portland Cement was a white Type I/II Portland Cement.

The "A" Lime was a white Type S hydrated lime.

The "A" Type S Masonry Cement was a white masonry cement with hydrated lime.

The "A" Type N Masonry Cement was a white masonry cement with hydrated lime.

The "B" Portland Cement was a gray Type I/II Portland Cement.

The "B" Lime was a white Type S hydrated lime.

The "B" Type S Masonry Cement was a gray masonry cement with no hydrated lime.

The "B" Type N Masonry Cement was a gray masonry cement with no hydrated lime.

Table 4.1 "A" Cementitious Material Specific Gravities and Manufacturer's Unit Weights

Material	Specific Gravity	Unit Weight (pcf)
"A" Portland Cement	3.041	94
"A" Hydrated Lime	2.396	40
"A" Type S Masonry Cement	2.861	75
"A" Type N Masonry Cement	2.838	70

Table 4.2 "B" Cementitious Material Specific Gravity and Manufacturer's Unit Weights

Material	Specific Gravity	Unit Weight (pcf)
"B" Portland Cement	3.145	94
"B" Hydrated Lime	2.401	40
"B" Type S Masonry Cement	2.946	75
"B" Type N Masonry Cement	2.893	70

4.2.2 Sands

Tables 4.3 and 4.4 show the sieve analysis results for the “A” sand and “B” sand respectively, which were from two different sources in Texas. “A” Sand was a manufactured sand; “B” Sand was a natural sand. Samples of each sand were sent to an independent testing laboratory, which measured the dry specific gravity of Sand “A” to be 2.625 and Sand “B” to be 2.626. The “A” sand complied with the gradation requirements of ASTM C 144 Sections 4.1 and 4.2, shown in Figure 1.2. The “B” sand did not comply with the gradation requirements of ASTM C 144 Sections 4.1 and 4.2. ASTM Section 4.4 permits the use of such sand “provided the mortar can be prepared to comply with the aggregate ratio, water retention, air content, and compressive strength requirements of the property specification of Specification C 270.” As shown in the rest of this chapter, the property testing performed as part of this research indicates that all “B” mortars complied with ASTM Section 4.4 except for the –MSB— laboratory mortar, which represents the Masonry Cement Type S “B” mortars. The –MSB— laboratory mortar only failed to comply because the average cube strength was less than 1800 psi.

Table 4.3 “A” Sand Gradation for Comparison with ASTM C 144 Sections 4.1 and 4.2*

Sieve Size	Percent Passing	Percent Retained on the Individual Sieve
No. 4	100.0	0
No. 8	99.2	1
No. 16	90.1	9
No. 30	73.9	16
No. 50	33.5	40
No. 100	8.2	25
No. 200	2.8	5

* Refer to Figure 1.2 for ASTM C 144 Requirements

Table 4.4 “B” Sand Gradation for Comparison with ASTM C 144 Sections 4.1 and 4.2*

Sieve Size	Percent Passing	Percent Retained on the Individual Sieve
No. 4	100.0	0
No. 8	99.9	0
No. 16	99.7	0
No. 30	98.6	1
No. 50	86.3	12
No. 100	22.4	64
No. 200	0.9	21

* Refer to Figure 1.2 for ASTM C 144 Requirements

4.2.3 Mortar Batch Designs

All test mortars were batched as shown in Tables 4.5 and 4.6 according to ASTM C 270 Table 1 for the Proportion Method of specifying mortar. Each batch contained 1.5 cubic feet of sand, measured in a one cubic foot box with a mark at half-height and stored in plastic bags. The corresponding volumes of cementitious materials were then measured by weight, using the manufacturer’s unit weights listed above to determine the required weight of each material. Each mortar batch design was also tested and determined to comply with ASTM C 270 Table 2 for the Property Method of specifying mortar, as shown in Table 4.7, except for the –MSB— mortar, which represents the Masonry Cement Type S “B” mortars. The –MSB— laboratory mortar only failed to comply because the average cube strength was less than 1800 psi.

Table 4.5 “A” Mortar Volumetric Proportions for Comparison with ASTM C 270 Table 1*

Mortar ID	“A” Portland Cement	“A” Hydrated Lime	“A” Type S Masonry Cement	“A” Type N Masonry Cement	“A” Sand
-PSA---	1	½	-	-	3
-PNA---	1	1 ¼	-	-	3
-MSA---	-	-	1	-	3
-MNA---	-	-	-	1	3

* Refer to Figure 1.1 for ASTM C 270 Table 1 Requirements

Table 4.6 “B” Mortar Volumetric Proportions for Comparison with ASTM C 270 Table 1*

Mortar ID	“B” Portland Cement	“B” Hydrated Lime	“B” Type S Masonry Cement	“B” Type N Masonry Cement	“B” Sand
-PSB---	1	½	-	-	3
-PNB---	1	1 ¼	-	-	3
-MSB---	-	-	1	-	3
-MNB---	-	-	-	1	3

* Refer to Figure 1.1 for ASTM C 270 Table 1 Requirements

Table 4.7 Laboratory Mortar Properties for Comparison with ASTM C 270 Table 2*

Mortar ID	Mortar Type	Avg. 28 Day Compressive Strength (psi)**	Water Retention (%)	Air Content (%)	Aggregate Ratio
-PSA---	S	3259	92	1.7	3
-PNA---	N	1393	95	1.7	3
-MSA---	S	2290	97	7.0	3
-MNA---	N	2055	85	3.0	3
-PSB---	S	2853	91	1.4	3
-PNB---	N	1455	85	4.6	3
-MSB---	S	1348	85	8.7	3
-MNB---	N	828	81	8.9	3

* Refer to Figure 1.1 for ASTM C 270 Table 2 Requirements
** Refer to Table 4.8 for Individual Cube Compressive Strengths

Table 4.8 ASTM C 270 Laboratory Mortar Cube Individual 28 Day Compressive Strengths

Mortar ID	1st Test (psi)	2nd Test (psi)	3rd Test (psi)	Average (psi)	Std. Dev. (psi)	C.O.V. (%)
-PSA---	3108	3283	3385	3259	72	2.2
-PNA---	1430	1374	1376	1393	1	0.1
-MSA---	2255	2308	2307	2290	1	0.0
-MNA---	2057	2003	2105	2055	72	3.5
-PSB---	2815	2704	3041	2853	238	8.4
-PNB---	1418	1465	1481	1455	11	0.8
-MSB---	1388	1422	1234	1348	133	9.9
-MNB---	805	836	844	828	6	0.7

4.2.4 Masonry Unit Properties

The “A” and “B” concrete masonry units were nominally 8 inches wide, 8 inches tall, and 16 inches long, from two different manufacturing plants in Texas. The manufacturers indicated that the units were lightweight units, conforming with ASTM C 90.

The “A” and “B” clay brick units were standard modular units, from two different sources. The manufacturers indicated that the units were hollow brick, conforming with ASTM C 652.

Three different shapes of “A” concrete masonry units were used in this research. The vast majority of prisms were constructed with units that had no indentations on either end, as shown in Figure 4.1, referred to herein as “Shape I”. However, the “A” Concrete Masonry Dry Limit Flexural Prisms were constructed with units having a small notch on one end, as shown in Figure 4.2, referred to herein as “Shape II”. And, the “A” Concrete Masonry Compression Prisms (for both Control and Wet Limit Conditions) were constructed with units that had an offset web on one end, as shown in Figure 4.3, referred to herein as “Shape III”.

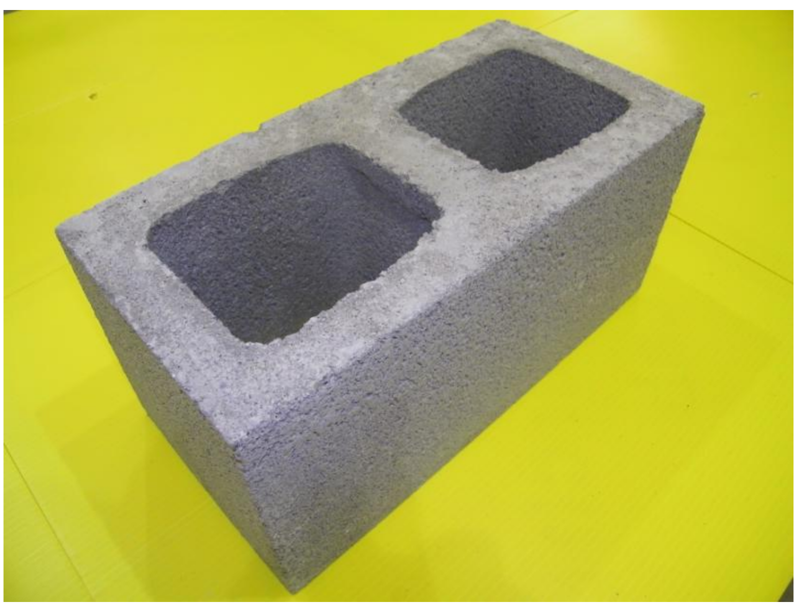

Figure 4.1 “A” Concrete Masonry Unit Shape I

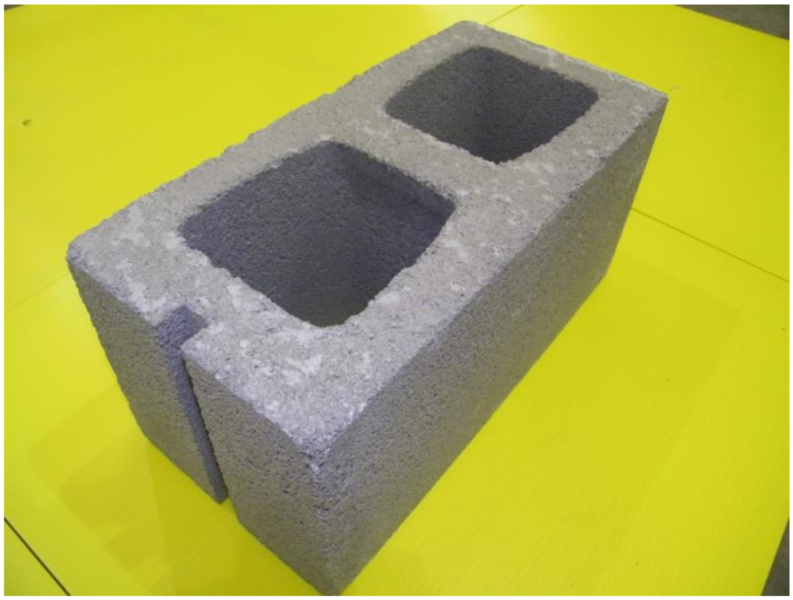

Figure 4.2 “A” Concrete Masonry Unit Shape II

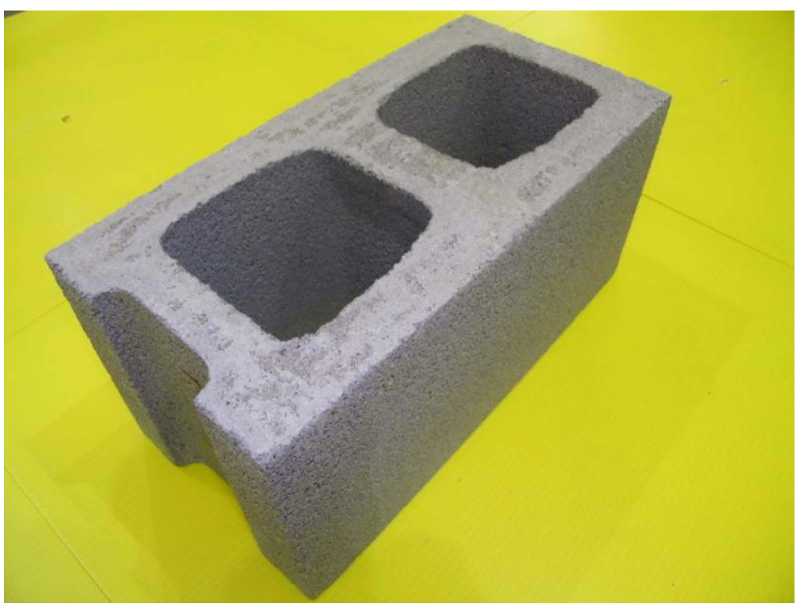

Figure 4.3 “A” Concrete Masonry Unit Shape III

Only one shape of each type of unit was used in prisms with “B” concrete masonry units, “A” clay brick masonry units, and “B” clay brick masonry units. For each of these categories, the only shape used is referred to as “Shape I” for that category. Refer to Figures 4.4, 4.5, and 4.6 for photographs of these shapes.

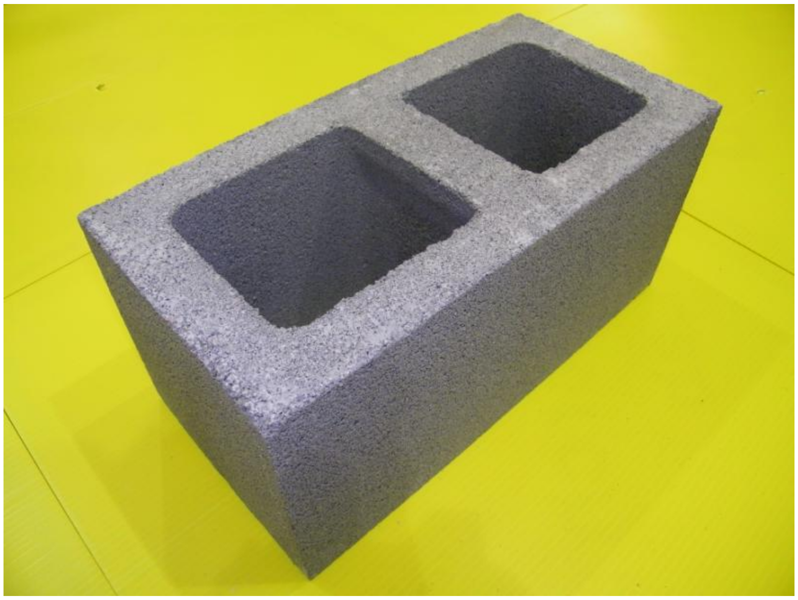

Figure 4.4 “B” Concrete Masonry Unit

Figure 4.5 “A” Clay Brick Unit

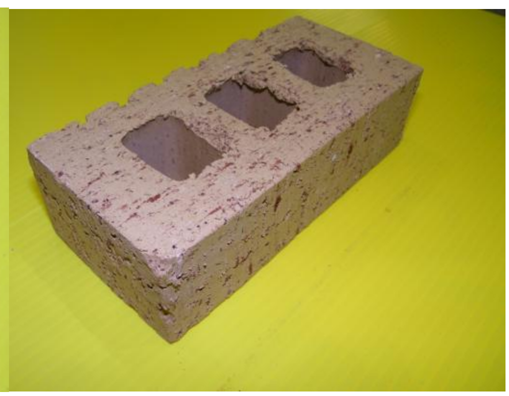

Figure 4.6 “B” Clay Brick Unit

The net areas of the concrete masonry units were measured per ASTM C 140. The void areas of the cores for the clay brick masonry units were measured per ASTM C 67, which requires filling the cores with sand to determine the volume of the cores. The gross areas of the clay brick units were measured directly. The net areas of the clay brick units were determined by subtracting the void areas of the cores and the measured void areas of scoring at the back of the clay brick units from the gross area. The percent solid for each brick was then calculated by dividing the net area by the gross area. Table 4.9 shows the net areas which were measured for three individual units that were chosen to be representative of each type of unit and were not used for any other testing. Table 4.10 shows the percent solid areas measured for the clay brick units. Units with less than 75% solid area are considered hollow. All units are hollow.

Table 4.9 Net Areas of Masonry Units

Material	Product Series	Shape ID	1st Unit (in^2)	2nd Unit (in^2)	3rd Unit (in^2)	Avg. (in^2)	Std. Dev. (in^2)	C.O.V. (%)
Concrete	“A”	I	62.9	62.7	62.8	62.8	0.1	0.1
Concrete	“A”	II	62.9	63.6	63.4	63.3	0.3	0.5
Concrete	“A”	III	63.6	63.5	63.6	63.6	0.0	0.1
Clay Brick	“A”	I	18.8	19.4	19.3	19.2	0.3	1.8
Concrete	“B”	I	59.3	59.5	58.7	59.2	0.4	0.7
Clay Brick	“B”	I	20.1	20.0	20.0	20.0	0.4	0.2

Table 4.10 Percent Solid Area of Clay Brick Masonry Units

Material	Product Series	Shape ID	1st Unit (%)	2nd Unit (%)	3rd Unit (%)	Avg. (%)	Std. Dev. (%)	C.O.V. (%)
Clay Brick	“A”	I	70.5	70.7	70.8	70.7	0.2	0.3
Clay Brick	“B”	I	73.4	73.3	73.2	73.3	0.1	0.1

The compressive strengths of representative masonry units, calculated using net areas and not gross areas, for each type of unit were capped with gypsum on a plexiglass capping table, tested and measured as shown in Table 4.11. For concrete masonry units, a single full-size unit was tested each time per ASTM C 140. For clay brick masonry units, a single unit was cut in half and one half was tested from each unit per ASTM C 67.

Table 4.11 Compressive Strengths of Masonry Units, Calculated Using Net Areas

Material	Series	Shape	1st Test (psi)	2nd Test (psi)	3rd Test (psi)	4th Test (psi)	5th Test (psi)	Avg. (psi)	Std. Dev. (psi)	C.O.V. (%)
C	"A"	I	2127	2370	1916	2308	2543	2253	240	10.6
C	"A"	II	1911	2537	2006	1963	1820	2047	282	13.8
C	"A"	III	3033	2783	2151	2933	2741	2728	343	12.6
B	"A"	I	19399	17133	18828	22118	16986	18893	2085	11.0
C	"B"	I	1970	2090	2185	2357	1952	2111	167	7.9
B	"B"	I	7923	7485	9656	8093	10466	8725	1272	14.6

Each of the units used for net area measurement were placed on a sheet of paper and traced with a pen so that the center of the pen was approximately 1/32 inch from the face of the unit being traced. In addition, a physical dimension of one side of the unit was measured using calipers and recorded. These tracings were electronically scanned and inserted into AutoCAD drawing files. In the AutoCAD drawing file: AutoCAD polylines were created along the lines of the scanned image; using the ALIGN command, these polylines were scaled to the recorded dimension; using the OFFSET command, a new set of polylines were created that were 1/32 inch away from the original polylines in the direction of the true surface measured, and the original polylines were deleted; using the ALIGN command again, these new polylines were scaled to the recorded dimension recorded; using the REGION command, regions were defined by the polylines; using the SUBTRACT command, void regions such as cores were subtracted from the solid regions; then, using the MASSPROP command, properties such as area, moment of inertia, and center of gravity location were automatically generated by AutoCAD and used to determine the section moduli for each shape, as shown in Table 4.12.

It is noteworthy that these section moduli are based on the net area of the clay brick units, with a reduction for the scoring at the back of the unit, not the gross areas. In addition, it is noteworthy that these section moduli are based on the smallest area of each concrete masonry unit, which occurs at the bottom of the unit as it is conventionally laid by masons into walls during construction. These section moduli were used to determine the flexural strength of prisms because the masons were instructed to fully bed all of the flexural specimens.

Table 4.12 Section Moduli of Masonry Units Tested in Flexure, Calculated Using Net Areas

Material	Product Series	Shape	1st Unit (in^3)	2nd Unit (in^3)	3rd Unit (in^3)	Avg. (in^3)	Std. Dev. (in^3)	C.O.V. (%)
Concrete	"A"	I	119.6	118.9	121.4	120.0	1.3	1.1
Concrete	"A"	II	119.8	120.6	124.3	121.6	2.4	2.0
Clay Brick	"A"	I	13.6	14.5	13.9	14.0	0.5	3.3
Concrete	"B"	I	117.1	119.4	122.0	119.5	2.5	2.1
Clay Brick	"B"	I	14.9	15.1	14.6	14.9	0.3	1.7

The oven-dried Initial Rate of Absorption (IRA) for full size samples of each type of brick were measured per ASTM C 67 and recorded as shown in Table 4.13 and Table 4.14. (These units were not used for any other testing.) This information is useful in evaluating the compatibility of a mortar water content with a unit's initial rate of absorption, which can have a significant influence on the flexural strength of unreinforced masonry.

Table 4.13 Oven-dried Initial Rate of Absorption (g/min/30 in.^{2}) for Clay Brick Units

Brick ID	1st Test	2nd Test	3rd Test	4th Test	5th Test	Average	C.O.V.(%)
"A" Brick	4.1	3.1	4.1	3.9	3.3	3.7	12.7
"B" Brick	23.7	25.4	24.9	25.4	21.9	24.3	6.1

Table 4.14 Ambient Air-dried Initial Rate of Absorption (g/min/30 in.^{2}) for Clay Brick Units

Brick ID	1st Test	2nd Test	3rd Test	4th Test	5th Test	Average	C.O.V.(%)
"A" Brick	1.4	3.1	2.3	2.5	2.7	2.4	26.4
"B" Brick	23.2	23.6	21.8	21.4	21.0	22.2	5.1

Before being installed in a prism, all masonry units were stored indoors for a minimum of 48 hours in a dry condition, with the exception of units used in prisms tested under wet limit conditions. All of the units used to test wet limit conditions were stored on pallets in a loose configuration in a room with 100% humidity at 75°F for a minimum of 7 days and were sprayed thoroughly with water to simulate a rain once a day during the two days before prism construction. In addition to the units which were used to construct prisms, additional units were exposed to the same wetting conditions for the purpose of weighing them during prism construction and oven-drying them afterwards to determine the moisture contents of the units as shown in Table 4.15. (These units were not used for any other testing.) For each product series, the first unit moisture content measurement was made when construction of the first wet limit condition prism began; the second measurement, between the two middle prisms; the third measurement, after the last prism.

Table 4.15 Moisture Contents of Wetted Masonry Units

Unit ID	1st Unit (%)	2nd Unit (%)	3rd Unit (%)	Avg. (%)	Std. Dev. (%)	C.O.V. (%)
C—AWC	7.0	6.0	6.5	6.5	0.5	7.9
C—AWF	7.3	7.8	7.8	7.6	0.3	4.0
B--AW-	2.0	2.1	0.7	1.6	0.8	50.2
C--BW-	6.0	5.5	5.4	5.7	0.3	6.2
B--BW-	6.4	6.4	6.7	6.5	0.1	2.1

4.3 Equipment

4.3.1 ASTM C 270 Property Method Testing Equipment

The mortar for each set of prisms was tested for compliance with the Property Method of ASTM C 270 to document mortar properties at the standardized flow. The small, prescribed batches were prepared in a Hobart mixer as shown in Figure 4.7 with a trial amount of water.

Figure 4.7 Hobart Mixer for Property Test Batches

After mixing per ASTM C 305, the mortar flow was then measured by using an ASTM C 230 flow table, as shown in Figures 4.8 and 4.9, with a motor that repetitively drops the table. Properties of mortars with a flow of 110 +/- 5 percent (percent increase in diameter) were measured to compare with ASTM C 270 Table 2 values.

Figure 4.8 Flow Table with Mold

Figure 4.9 Flow Table after 25 Drops

Three 2 in. mortar cubes for were prepared using brass mortar cube gang molds as shown in Figure 4.10 and tested in a 60 kip compression machine as shown in Figure 4.11.

Figure 4.10 Brass Mortar Cube Mold

Figure 4.11 60 kip Compression Machine for Cube Compression Testing

A sample of each mortar was placed on an apparatus that creates a prescribed suction as shown in Figure 4.12 according to ASTM C 1506. The flow was then measured after the suction was applied for the prescribed amount of time. And, then the percent water retention was calculated by dividing the flow after suction by the flow before suction.

Figure 4.12 Suction Apparatus for the Water-Retention Test

The air content of the mortar was determined by measuring the weight of mortar in a 400 mL brass container and then using the method prescribed in ASTM C 270 which uses the weights and densities of all materials used in the mortar. (The densities of the materials for this research were determined by an independent testing laboratory.)

4.3.2 Net Area Measurement Equipment

The void areas of the cores in the clay brick units were calculated per ASTM C 67, using the "B" sand to fill in the cores and determine the volume of the cores and dividing this volume by the unit height. The net areas for the clay brick units were calculated by subtracting the areas of the cores and the measured areas of the notches at the back of the unit as shown in Figures 4.5 and 4.6 from the gross area of the unit.

The net areas of the concrete masonry units were calculated per ASTM C 140, which uses the immersed weight, saturated weight and oven-dry weight of the unit. To determine the immersed weight, a scale with a hook underneath was suspended over a tank of water with a hanging wire that was wrapped around representative units, as shown in Figure 4.13. The units were oven dried using an industrial heating unit, shown in Figure 4.14.

Figure 4.13 Immersed Weight Measurement

Figure 4.14 Industrial Heating Unit

4.3.3 Testing Program Equipment

The mortar for each set of prisms constructed was batched with 1 1/2 cubic feet of sand, measured using a 1 cubic foot plywood box as shown in Figure 4.15. The required volumes of cementitious materials to be proportionate with 1 1/2 cubic feet of sand were determined by Tables 4.5 and 4.6 for each batch. The required weights of cementitious materials were calculated based on the required volumes and the unit weights shown in Tables 4.1 and 4.2. The cementitious materials were then measured by weight on a scale that was accurate to the nearest 0.05 pounds, as shown in Figure 4.16.

Figure 4.15 Plywood Box for Sand

Figure 4.16 Scale for Cementitious Materials

The materials for each batch were stored in plastic construction bags that were tied closed. For all of the masonry prisms, professional masons employed on commercial construction projects in the North Texas area operated the 6 cubic yard mechanical mixer, added materials to the mixer as shown in Figure 4.17, added water and constructed the prisms in bags at the curing location. The "A" Mason had approximately 25 years experience; the "B" Mason, approximately 30 years. All mortar water contents were established by the masons, after being introduced to the units, with the direction to prepare mortar that is "typical" for the control condition, "the wettest you would accept from your crew" for the wet limit condition, and "what you would want for repair or detailed work" for the dry limit condition. The mortar water content under the "dry limit" condition complied with the ASTM C 270 Section 7.3 requirement for a “maximum amount of water to produce a workable consistency” because the mason was directed to prepare mortar for repair or detailed work which required a stiff mortar.

Figure 4.17 Professional Mason Operating Mechanical Mixer

The batches were prepared on a schedule that staggered mixing operations to efficiently utilize each mason’s time yet also allow for testing of fresh mortar properties and construction of each set of prisms at the anticipated mortar age for placement. After mixing, each batch of mortar was placed in a clean wheelbarrow with an identification tag on the handle as shown in Figure 4.18. A separate timer for each wheelbarrow was started when the mortar was placed in the wheelbarrow to monitor mortar age. One shovel remained with each wheelbarrow for periodic mixing by hand to prevent segregation of the mortar.

Figure 4.18 Wheelbarrows with Identification Tags on Handles

Before prism construction, several fresh mortar properties were measured and recorded. The mortar flow was measured using the flow table show in Figures 4.8 and 4.9. The cone penetrometer shown in Figures 4.19 and 4.20 was used to document consistency by measuring and recording the depth of the cone penetration in mm. The mortar air content was measured using a pressuremeter as shown in Figure 4.21. In addition, cubes were made and tested using the same equipment as the ASTM C 270 Property Method tests described above.

Figure 4.19 Cone at Initial Setting

Figure 4.20 Cone Penetration after Release

Figure 4.21 Air Content by Pressuremeter

A 26 ft x 16 ft environmental chamber capable of continuous monitoring and automatic adjustment of temperature (36°F to 120°F operating range) and humidity, shown in Figure 4.22, was used to control the temperature of prisms, during the entire 28 day curing period.

Figure 4.22 Curing Chamber

As shown in Figure 4.23, the masons used trowels to fully mortar the bedjoints and strike the joints flush. All prisms were constructed in plastic bags that were closed with ties after each prism was constructed, as shown in Figure 4.24.

Figure 4.23 Mason Constructing Prisms

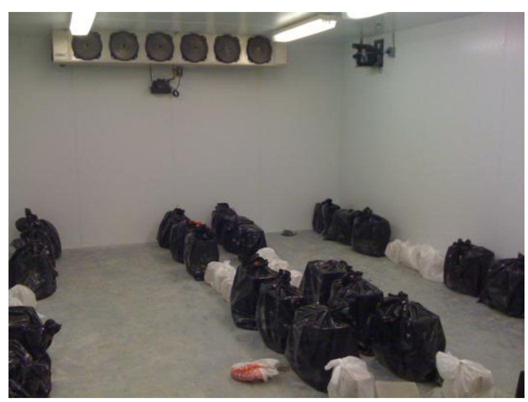

Figure 4.24 Prisms Cured in Plastic Bags

As shown in Figure 4.25, all compression prisms (concrete and clay brick) were capped with a gypsum cement compound on a level plexiglass surface capping table.

Figure 4.25 Capped Compression Prisms

As shown in Figure 4.26, all compression prisms were tested in a 400 kip compression machine with steel platens that comply with ASTM C 1314 for full 8X8X16 concrete units.

Figure 4.26 400 kip Compression Machine

As shown in Figure 4.27, each bedjoint of all clay brick flexural prisms were tested in flexure using a conventional hydraulic bond wrench machine.

Figure 4.27 Clay Brick Bond Wrench Machine

As shown in Figure 4.28, all concrete flexural prisms were tested in flexure by 2 people manually pouring scoops of steel shot into a bucket suspended by a hook at the end of a steel upper clamp bracket, using a timer to comply with ASTM C 1072.

Figure 4.28 Concrete Bond Wrench Testing

CHAPTER 5

TEST DATA

As described in the preceding chapters, professional masons constructed all masonry prisms with fully bedded 3/8 inch thick bedjoints. The shape and orientation of all units in each prism were identical. All concrete masonry prisms were made two units high, without reducing the unit sizes. All concrete masonry prisms were constructed so that the narrow edge of the webs and face shells were on the bottom of each unit. Clay brick masonry prisms were made three units high for compression testing and four units high for flexural testing. Clay brick masonry prisms were tested in flexure with the flat face of the units in compression. The mortar cubes were cured under the same temperature conditions as the masonry prisms. The results of the testing program are provided in Tables 5.1 to 5.12. Considering the calculated coefficients of variation, a normal distribution was assumed for compression strengths and a log normal distribution for flexural strengths. The characteristic values represent the 95% fractile.

Table 5.1 “A” Mortar Properties at Time of Initial Placement

Mortar ID	Approx. Mortar Age (minutes)	Flow (%)	Cone Penetration (mm)	Air Content (%)
CPSACC	15	106	54	2.4
CPSACF	15	113	53	3.4
BPSAC-	15	110	50	3.5
CPSAWC	120	117	72	1.0
CPSAWF, BPSAWC, & BPSAWF	120	116	66	1.2
-PSAD-	120	98	50	3.6

Table 5.1 - continued

Mortar ID	Approx. Mortar Age (minutes)	Flow (%)	Cone Penetration (mm)	Air Content (%)
CPNACC	15	107	52	2.8
CPNACF	15	101	41	4.2
BPNAC-	15	109	50	3.3
CPNAWC	120	127	71	0.7
CPNAWF, BPNAWC, & BPNAWF	120	123	72	0.7
-PNAD-	120	83	39	3.8
CMSACC	15	111	44	9.0
CMSACF	15	100	42	8.0
BMSAC-	15	103	42	8.9
CMSAWC	120	139	75	9.6
CMSAWF, BMSAWC, & BMSAWF	120	142	69	7.7
-MSAD-	120	113	50	9.2
CMNACC	15	125	54	5.3
CMNACF	15	126	47	6.5
BMNAC-	15	123	42	6.6
CMNAWC	120	131	73	4.5
CMNAWF, BMNAWC, & BMNAWF	120	129	70	3.1
-MNAD-	120	103	42	6.0

Table 5.2 "A" Mortar Cube 28 Day ASTM C 780 Compressive Strengths

Mortar ID	1st Test (psi)	2nd Test (psi)	3rd Test (psi)	Average (psi)	C.O.V. (%)	Characteristic Value (psi)
CPSACC	2219	2327	2276	2274	2.4	2185
CPSACF	3017	2450*	3225	2897	13.8	2235
BPSAC-	3275	3476	3457	3403	3.3	3220
CPSAWC	1385	1454	1385	1408	2.8	1342
CPSAWF, BPSAWC, & BPSAWF	2189	2032	2130	2117	3.7	1987
-PSAD-	3213	2903	3099	3072	5.1	2813
CPNACC	2329	2253	2412	2331	3.4	2201
CPNACF	1417	1562	1679	1553	8.5	1337
BPNAC-	1446	1475	1515	1479	2.4	1421
CPNAWC	1256	1314	1358	1309	3.9	1225
CPNAWF, BPNAWC, & BPNAWF	583	479*	571	544	10.5	450
-PNAD-	1361	1462	1408	1410	3.6	1326
CMSACC	1946	1995	1926	1955	1.8	1897
CMSACF	1979	1765	1891	1878	5.7	1700
BMSAC-	1947	1816	1734	1832	5.9	1655
CMSAWC	1895	1893	1924	1904	0.9	1876
CMSAWF, BMSAWC, & BMSAWF	1608	1518	1599	1575	3.2	1493
-MSAD-	2178	2298	2419	2298	5.2	2100

* Instead of discarding this test result per ASTM C 780 Appendix A6.9.1, this test was included in calculating the average so as to conservatively evaluate CSA A179 field mortar requirements.

Table 5.2 - continued

Mortar ID	1st Test (psi)	2nd Test (psi)	3rd Test (psi)	Average (psi)	C.O.V. (%)	Characteristic Value (psi)
CMNACC	2250	2201	2292	2248	2.0	2174
CMNACF	1809	1989	1954	1917	5.0	1760
BMNAC-	1846	1944	1924	1905	2.7	1819
CMNAWC	1596	1672	1676	1648	2.7	1574
CMNAWF, BMNAWC, & BMNAWF	2603	1769*	2095	1975	9.1	1678
-MNAD-	2731	2764	2666	2720	1.8	2639

* Instead of discarding this test result per ASTM C 780 Appendix A6.9.1, this test was included in calculating the average so as to conservatively evaluate CSA A179 field mortar requirements.

Table 5.3 “B” Mortar Properties at Time of Initial Placement

Mortar ID	Approx. Mortar Age (minutes)	Flow (%)	Cone Penetration (mm)	Air Content (%)
CPSBCC	15	134	53	3.2
CPSBCF	15	135	52	3.5
BPSBC-	15	143	50	3.8
-PSBW-	120	141	74	2.1
-PSBD-	120	111	35	3.7
CPNBCC	15	134	55	3.0
CPNBCF	15	124	52	4.0
BPNBC-	15	129	55	3.6
-PNBW-	120	138	72	2.5
-PNBD-	120	103	28	4.6
CMSBCC	15	>150*	73	8.5
CMSBCF	15	116	58	12.7
BMSBC-	15	123	61	12.5
-MSBW-	120	>150**	72	11.0
-MSBD-	120	109	45	11.5
CMNBCC	15	150	64	9.2
CMNBCF	15	126	58	13.5
BMNBC-	15	123	62	13.0
-MNBW-	120	>150**	72	11.1
-MNBD-	120	108	42	11.1

*Mortar flowed off the table after 16 drops out of the required 25.
**Mortar flowed off the table after 21 drops out of the required 25.

Table 5.4 “B” Mortar Cube 28 Day ASTM C 780 Compressive Strengths

Mortar ID	1st Test (psi)	2nd Test (psi)	3rd Test (psi)	Average (psi)	C.O.V. (%)	Characteristic Value (psi)
CPSBCC	-*	3356	3256	3306	2.1	3189
CPSBCF	2508	2190	2592	2430	8.7	2080
BPSBC-	2766	2669	2915	2783	4.5	2578
-PSBW-	2317	2451	2454	2407	3.2	2278
-PSBD-	3181	3204	3112	3165	1.5	3086
CPNBCC	1229	1253	1199	1227	2.2	1182
CPNBCF	1061	995	1091	1049	4.7	968
BPNBC-	1067	1019	1021	1036	2.6	991
-PNBW-	1120	1221	1209	1183	4.6	1092
-PNBD-	1129**	1386	1460	1325	13.1	1040
CMSBCC	-*	1720	-*	1720	-*	-*
CMSBCF	1717	1822	1643	1727	5.2	1579
BMSBC-	1732	1622	1682	1679	3.3	1588
-MSBW-	2053	2170	2074	2099	3.0	1997
-MSBD-	1388	1324	1340	1351	2.5	1297
CMNBCC	960	983	1110	1017	7.9	883
CMNBCF	1159	1117	1140	1139	1.8	1104
BMNBC-	1049	996	1040	1028	2.7	982
-MNBW-	895	884	859	879	2.1	849
-MNBD-	1006	1170	1071	1082	7.6	945

* Data not available. Error during loading.
** Instead of discarding this test result per ASTM C 780 Appendix A6.9.1, this test was included in calculating the average so as to conservatively evaluate CSA A179 field mortar requirements.

Table 5.5 “A” Concrete Masonry Prism 28 Day Net Area ASTM C 1314 Compressive Strengths

Mortar ID	1st Test (psi)	2nd Test (psi)	3rd Test (psi)	Average (psi)	C.O.V. (%)	Characteristic Value (psi)
CPSACC-	2299	2199	2639	2379	9.7	2000
CPSAWC-	2236	2501	2174	2304	7.5	2019
CPNACC-	2548	3001	2721	2757	8.3	2381
CPNAWC-	2379	2539	1939	2286	13.6	1773
CMSACC-	2156	2528	2388	2357	8.0	2047
CMSAWC-	2061	2063	2119	2081	1.6	2027
CMNACC-	2326	2425	2149	2300	6.1	2069
CMNAWC-	2308	2326	2443	2359	3.1	2239

Table 5.6 “B” Concrete Masonry Prism 28 Day Net Area ASTM C 1314 Compressive Strengths

Mortar ID	1st Test (psi)	2nd Test (psi)	3rd Test (psi)	Average (psi)	C.O.V. (%)	Characteristic Value (psi)
CPSBCC-	2381	2943	1999	2441	19.5	1657
CPSBWC-	1686	1708	2022	1805	10.4	1495
CPNBCC-	2211	1812	1969	1997	10.1	1665
CPNBWC-	1757	1375	1576	1569	12.2	1254
CMSBCC-	2063	2220	2372	2218	7.0	1964
CMSBWC-	1503	1966	1764	1744	13.3	1361
CMNBCC-	2043	1927	2084	2018	4.1	1883
CMNBWC-	1880	1768	1387	1679	15.4	1253

Table 5.7 “A” Clay Brick Masonry Prism 28 Day Net Area ASTM C 1314 Compressive Strengths

Mortar ID	1st Test (psi)	2nd Test (psi)	3rd Test (psi)	Average (psi)	C.O.V. (%)	Characteristic Value (psi)
BPSACC-	7096	9132	7695	7975	13.1	6249
BPSAWC-	4391	4307	4681	4460	4.4	4137
BPNACC-	4691	5780	5961	5477	12.5	4343
BPNAWC-	5249	5813	5818	5627	5.8	5087
BMSACC-	3929	5399	4859	4729	15.7	3501
BMSAWC-	5720	5817	7536	6358	16.1	4673
BMNACC-	5391	5666	5667	5575	2.8	5313
BMNAWC-	6278	5667	4814	5587	13.2	4374

Table 5.8 “B” Clay Brick Masonry Prism 28 Day Net Area ASTM C 1314 Compressive Strengths

Mortar ID	1st Test (psi)	2nd Test (psi)	3rd Test (psi)	Average (psi)	C.O.V. (%)	Characteristic Value (psi)
BPSBCC-	5215	5015	4140	4790	11.9	3846
BPSBWC-	3648	3734	4416	3393	10.7	2698
BPNBCC-	2608	3422	3520	3184	15.7	2357
BPNBWC-	3501	3086	3123	3237	7.1	2858
BMSBCC-	4040	4342	4052	4145	4.1	3863
BMSBWC-	3764	3676	3842	3761	2.2	3624
BMNBCC-	4227	3783	3819	3943	6.3	3535
BMNBWC-	2774	2806	2579	2719	4.5	2516

Table 5.9 “A” Concrete Masonry Prism 28 Day Net Area ASTM C 1072 Flexural Strengths

Mortar ID	1st Test (psi)	2nd Test (psi)	3rd Test (psi)	Average (psi)	C.O.V. (%)	Characteristic Value (psi)
CPSACF-	33	31	54	39	32.0	23
CPSAWF-	49	51	75	59	24.6	39
CPSADF-	33	28	26	29	11.9	24
CPNACF-	25	40	48	38	31.5	21
CPNAWF-	25	23	15	21	25.0	13
CPNADF-	16	18	23	19	20.7	14
CMSACF-	63	68	61	64	5.4	58
CMSAWF-	34	47	43	41	15.3	31
CMSADF-	1	21	24	15	81.7	0
CMNACF-	50	64	37	50	26.5	31
CMNAWF-	41	24	26	30	30.9	18
CMNADF-	8	8	6	7	14.0	6

Table 5.10 "B" Concrete Masonry Prism 28 Day Net Area ASTM C 1072 Flexural Strengths

Mortar ID	1st Test (psi)	2nd Test (psi)	3rd Test (psi)	Average (psi)	C.O.V. (%)	Characteristic Value (psi)
CPSBCF-	68	60	86	71	18.6	52
CPSBWF-	25	61	49	45	40.7	20
CPSBDF-	28	28	26	27	5.1	25
CPNBCF-	11	12	1	8	76.5	0
CPNBWF-	50	27	37	38	30.0	22
CPNBDF-	24	21	17	21	15.2	15
CMSBCF-	48	55	34	46	23.4	30
CMSBWF-	39	14	27	27	47.8	10
CMSBDF-	8	13	10	10	27.8	7
CMNBCF-	37	63	49	49	26.3	31
CMNBWF-	10	5	14	10	43.0	4
CMNBDF-	10	12	11	11	9.3	9

Table 5.11 “A” Clay Brick Prism 28 Day Net Area ASTM C 1072 Flexural Strengths

Mortar ID	1st Test (psi)	2nd Test (psi)	3rd Test (psi)	Average (psi)	C.O.V. (%)	Characteristic Value (psi)
BPSACF-	131	122	103	119	12.0	96
BPSAWF-	79	83	87	83	4.8	77
BPSADF-	78	91	68	79	14.9	62
BPNACF-	79	89	97	88	10.1	74
BPNAWF-	51	50	51	51	1.4	50
BPNADF-	88	76	69	78	12.4	63
BMSACF-	133	115	144	131	11.4	108
BMSAWF-	91	60	116	89	31.4	50
BMSADF-	118	94	55	89	35.5	44
BMNACF-	136	149	118	134	11.6	110
BMNAWF-	89	118	99	102	14.6	80
BMNADF-	114	79	48	80	40.9	37

Table 5.12 “B” Clay Brick Prism 28 Day Net Area ASTM C 1072 Flexural Strengths

Mortar ID	1st Test (psi)	2nd Test (psi)	3rd Test (psi)	Average (psi)	C.O.V. (%)	Characteristic Value (psi)
BPSBCF-	250	159	88	166	49.0	64
BPSBWF-	202	210	93	168	38.9	74
BPSBDF-	40	35	67	47	36.7	26
BPNBCF-	124	113	102	113	9.6	96
BPNBWF-	91	93	60	81	22.8	53
BPNBDF-	46	89	30	55	55.5	20
BMSBCF-	93	164	93	116	35.2	65
BMSBWF-	134	150	137	141	6.0	127
BMSBDF-	84	73	25	61	51.5	18
BMNBCF-	58	104	94	85	27.8	49
BMNBWF-	77	91	74	81	11.3	67
BMNADF-	54	116	56	75	47.0	35

CHAPTER 6

ANALYSIS

6.1 Analysis of 28 Day Compression Prism Results

The prism compressive strengths were measured in accordance with ASTM C 1314, the industry standard method of evaluating compressive strength of masonry. Tables 6.1 and 6.2 show characteristic values representing the 95% fractile for compression strengths of concrete and clay brick masonry. For Allowable Stress Design (ASD) and Strength Design (SD), TMS 602-08 Tables and 1 and 2 permit assumed values for the compressive strength of masonry according to the Unit Strength Method based on the unit material (concrete or clay), unit strength and mortar type (M, S or N) as shown for the average unit strengths.

Table 6.1 Summary of Characteristic Values for Concrete Prism Compression Strength*

Mortar ID	Average Unit Strength (psi)	TMS 602-08 Table 2 f'_m (psi)	Control Characteristic Value (psi)	Wet Characteristic Value (psi)	Wet/Control (%)
CPSA-C-	2728	1960	2000	2019	101
CPNA-C-	2728	1821	2381	1773	74
CMSA-C-	2728	1960	2047	2027	99
CMNA-C-	2728	1821	2069	2239	108
CPSB-C-	2111	1617	1657	1495	90
CPNB-C-	2111	1477	1665	1254	75
CMSB-C-	2111	1617	1964	1361	69
CMNB-C-	2111	1477	1883	1253	67

* Refer to Figure 1.3 for f'_m values from the Unit Strength Method

Table 6.2 Summary of Characteristic Values for Clay Brick Prism Compression Strength*

Mortar ID	Average Unit Strength (psi)	TMS 602-08 Table 1 f'_m (psi)	Control Characteristic Value (psi)	Wet Characteristic Value (psi)	Wet/Control (%)
BPSA-C-	18893	4000	6249	4137	66
BPNA-C-	18893	3000	4343	5087	117
BMSA-C-	18893	4000	3501	4673	133
BMNA-C-	18893	3000	5313	4374	82
BPSB-C-	8725	3144	3846	2698	70
BPNB-C-	8725	2616	2357	2858	121
BMSB-C-	8725	3144	3863	3624	84
BMNB-C-	8725	2616	3535	2516	71

* Refer to Figure 1.3 for f'_m values from the Unit Strength Method

The wet characteristic value divided by the control characteristic value, shown in Tables 6.1 and 6.2, indicates the combined influence of the many conditions tested to simulate the field condition limits permitted by TMS 402 and referenced standards. For concrete prisms, this relative strength was as low as 67% and as high as 108%, with an average of 85%, standard deviation of 16%, and a coefficient of variation of 18.8%. For clay brick prisms, this relative strength was as low as 70% and as high as 133%, with an average of 93%, standard deviation of 26%, and a coefficient of variation of 28.5%.

Some of the characteristic values, for control and wet conditions, were less than the values for f'_m permitted by the Unit Strength Method for ASD and SD. And, some of the values for simulated field mortar cubes were below the values in ASTM C 270 Table 2. However, the different design methods permitted by TMS 402 do not permit design compressive stresses to actually reach f'_m, which provides further safety. The factor of safety for compression associated with use of the Unit Strength Method for the characteristic values can be determined for each of the different design methods permitted by TMS 402-08.

6.1.1 Factors of Safety for Compression using Allowable Stress Design

For Allowable Stress Design (ASD) without the one-third stress increase, the factor of safety for compressive stress in the axial compression and flexure equations from TMS 402-08 for both unreinforced and reinforced masonry is the characteristic value multiplied by 4.0 and divided by f'_m , according to TMS 402 Commentary Sections 2.2.3.1 and 2.3.3.2. With the one-third stress increase, the factor of safety is the characteristic value multiplied by 3.0 and divided by f'_m. Refer to Table 6.3 which shows all factors of safety to be greater than 2.5.

6.1.2 Factors of Safety for Compression using Strength Design

For Strength Design (SD), the factor of safety is directly proportional to the load factor, which varies depending on the nature of the loading (dead, live, wind, etc…). One of the most critical load cases to consider with regard to factor of safety for compression, therefore, is the dead-load-only case with a load factor of 1.4 as required by ASCE 7-05 (33). The factor of safety is inversely proportional to the strength reduction factor, which varies in TMS 402-08 according to whether or not the masonry is reinforced: According to TMS 402-08 Section 3.1.4, the strength reduction factor, ϕ, for axial loads is 0.6 for unreinforced masonry and 0.9 for reinforced masonry. Therefore, the reinforced masonry is the more critical case to consider. Using the TMS 402-08 SD method, therefore, the dead-load-only case factor of safety for compressive stress associated with the axial compression and flexure equations from TMS 402-08 for reinforced masonry is the reported characteristic value multiplied by a load factor of 1.4, divided by the product of the strength reduction factor of 0.9 and f'_m . Refer to Table 6.4 which shows all factors of safety to be greater than 1.3.

Table 6.3 Measured Factors of Safety for Compression using ASD

Mortar ID	Control F.S. without 1/3 increase	Control F.S. with 1/3 increase	Wet F.S. without 1/3 increase	Wet F.S. with 1/3 increase
CPSA-C-	4.1	3.1	4.1	3.1
CPNA-C-	5.2	3.9	3.9	2.9
CMSA-C-	4.2	3.1	4.1	3.1
CMNA-C-	4.5	3.4	4.9	3.7
CPSB-C-	4.1	3.1	3.7	2.8
CPNB-C-	4.5	3.4	3.4	2.5
CMSB-C-	4.9	3.6	3.4	2.5
CMNB-C-	5.1	3.8	3.4	2.5
BPSA-C-	6.2	4.7	4.1	3.1
BPNA-C-	5.8	4.3	6.8	5.1
BMSA-C-	3.5	2.6	4.7	3.5
BMNA-C-	7.1	5.3	5.8	4.4
BPSB-C-	4.9	3.7	3.4	2.6
BPNB-C-	3.6	2.7	4.4	3.3
BMSB-C-	4.9	3.7	4.6	3.5
BMNB-C-	5.4	4.1	3.8	2.9

Table 6.4 Measured Factors of Safety for Compression using SD with a Dead-Load-Only case and Reinforced Masonry

Mortar ID	Control F.S.	Wet F.S.
CPSA-C-	1.6	1.6
CPNA-C-	2.0	1.5
CMSA-C-	1.6	1.6
CMNA-C-	1.8	1.9
CPSB-C-	1.6	1.4
CPNB-C-	1.8	1.3
CMSB-C-	1.9	1.3
CMNB-C-	2.0	1.3
BPSA-C-	2.4	1.6
BPNA-C-	2.3	2.6
BMSA-C-	1.4	1.8
BMNA-C-	2.8	2.3
BPSB-C-	1.9	1.3
BPNB-C-	1.4	1.7
BMSB-C-	1.9	1.8
BMNB-C-	2.1	1.5

6.1.3 Factors of Safety for Compression using Empirical Design

Empirical Design requires that the thickness of the masonry be a minimum of 6 inches. This analysis will only evaluate the factors of safety for the concrete masonry, which had units that were nominally 8 inches thick. The clay brick units would have to occur in a multi-wythe wall to be permitted by Empirical. This analysis is still relevant because multi-wythe clay brick masonry is generally not as common today in the construction industry as single wythe 8 inch concrete masonry construction.

TMS 402-08 Table 5.4.2 provides gross area compressive stress limits due to dead and live load. Table 6.5 provides a summary of the gross area characteristic values representing the 95% fractile prism compressive strengths from this research in a similar manner as the information presented in Tables 6.1 and 6.2. For Empirical Design (Empirical), 100% of the gross area characteristic values, for both the control and the wet conditions, were greater than the allowable gross area compressive stresses permitted by TMS 402-08 Table 5.4.2 for Empirical Design. However, note that the allowable gross compressive stresses are associated only with vertical dead and live loads, excluding other loads such as wind and seismic. The permitted values from TMS 402-08 Table 5.4.2 provided in Table 6.5 for both concrete and clay masonry are for hollow load bearing units, considering that the percent solid area was less than 75% of the gross area for all units used in this research.

For Empirical Design (Empirical), the factor of safety for compressive stress is the gross area characteristic value divided by the gross area compressive stresses permitted by TMS 402-08 Chapter 5 on Empirical Design.

Because the TMS 402-08 Table 5.4.2 values only limit vertical dead and live loads, additional compressive stresses may occur as a result of other loads such as wind and seismic. The compressive stresses in a wall will vary depending on the geometric arrangement of building elements and the loading under consideration. TMS 402-08 Section 5.1.2.2 does not permit the use of Empirical as part of the lateral force resisting system in Seismic Design

Table 6.5 Gross Area Characteristic Values for Concrete Prism Compression Strength

Mortar ID	Average Gross Area Unit Strength (psi)	TMS 402-08 Table 5.4.2 (psi)	Gross Area Control Characteristic Value (psi)	Gross Area Wet Characteristic Value (psi)
CPSA-C-	1457	112	1068	1078
CPNA-C-	1457	97	1271	947
CMSA-C-	1457	112	1093	1082
CMNA-C-	1457	97	1105	1196
CPSB-C-	1049	79	824	743
CPNB-C-	1049	73	828	693
CMSB-C-	1049	79	976	676
CMNB-C-	1049	73	936	623

Categories B and C, and does not permit the use of Empirical at all in Seismic Design Categories D, E and F. This analysis will consider the wind-load-only case with a single story, load bearing exterior wall, which may be used for comparison with the ASD and SD factors of safety provided above; the height to thickness ratio permitted for this case is 18 since the masonry in this research was unreinforced and ungrouted, which is classified as "other than solid". Because the concrete masonry units tested in this research had an 8 inch nominal thickness, this analysis will consider the maximum vertical span permitted by Empirical for the above case, which is 12 feet. TMS 402-08 Table 5.3.1 permits a maximum length-to-width ratio of 5:1 for cast-in-place concrete roof diaphragms, which is a critical case to consider when evaluating factors of safety in compression. TMS 402-08 Section 5.3.1.1 requires that shear wall segments which are relied upon as part of the lateral force resisting system must have a plan length that is at least one-half the wall height. Therefore, this analysis will consider the critical case with a symmetrical, rectangular, single-story building with only one load-bearing shear wall on each side that is 6 feet long and 12 feet tall, separated by a cast-in-place concrete diaphragm that is 30 feet long, such as a water treatment plant for a city may use to house control systems and water treatment equipment that is hung from the roof. This geometric arrangement of elements complies with TMS 402-08 Section 5.3.1.1, which requires the plan length of shear walls provided on each side to be a minimum of 0.2 times the long dimension of the building. The critical case to consider is if all of the roof load is placed upon the 6 ft long walls at each end. This could easily occur if the long walls are only bolted to the roof diaphragm by steel angles with vertically slotted holes, such as may be used if the city in this theoretical case wanted to increase the capacity of the water treatment plant as the city grows, so that portions of the long walls can be easily removed in the future. This analysis will assume the critical case in which the long walls are not load-bearing and that there is a separation joint at each corner so the long walls are not connected to the shear walls being analyzed. This is also a critical case because a portion of the long walls would otherwise act as flanges and reduce the compressive stresses in the shear walls being analyzed. This analysis will consider the cast-in-place concrete roof to be completely flat, assuming that tapered insulation will slope the roof for drainage. To conceal the roof slab and tapered insulation, this analysis will assume a parapet extends 2 ft 8 in. above the slab bearing elevation, which is 12 ft above the floor slab. According to TMS 402-08 Table 5.1.1, Empirical is permitted for the design of masonry elements that are part of the lateral force-resisting system of buildings that are less than 35 feet high in Basic Wind Speeds up to 110 mph. According to ASCE 7-05, referenced by TMS 402-08 Section 1.7, the wind load required for design of buildings may be calculated by either the Analytical Method or the Simplified Method. According to ASCE 7-05 Figure 6-2 of the Simplified Method, for the case where wind direction is parallel to the short walls, the net main-wind-force-resisting system wind pressure on the long walls for a flat roof in a 110 mph Basic Wind Speed is 19.2 psf in Zone A (end zone of wall) and 12.7 psf in Zone C (interior zone of wall). For Exposure Category C, these values are to be multiplied by 1.21. For an essential building, such as a Water Treatment Plant, these values are to be further increased by 1.15.

Therefore, this analysis will consider the case with both conditions, so that the wind pressure for design is 26.7 psf in Zone A and 17.7 psf in Zone C. The net uniform wind load into the roof diaphragm along the top of the long wall as determined by statics is therefore 239 plf for Zone A and 159 plf for Zone C. According to ASCE 7-05 Figure 6-2, the width of Zone A for the structure being analyzed is 6 feet. Therefore, the lateral force from the roof diaphragm transferred into the top of each shear wall is 2,865 pounds. Because the roof diaphragm is 12 feet above the foundation, the in-plane moment at the base of each cantilevered shear wall is therefore 412,600 pound-in. The in-plane bending section modulus of an unreinforced, ungrouted 6 ft long wall with fully mortared bedjoints as were used in this research, based on the measurements recorded as part of this research, would be 3,614 $in.^3$ for “A” units and 3,379 $in.^3$ for “B” units. Therefore, the maximum net area flexural stresses (both compression and tension) due to in-plane flexure from resisting the wind load would be 114 psi for “A” units and 122 psi for “B” units. The critical net area compressive stress permitted by Empirical for this structure is the sum of this flexural stress and the compressive stress due to axial dead and live loads. The maximum permitted gross area compressive stress due to axial dead and live loads will vary as shown in Table 6.5. The maximum permitted net area compressive stress for each unit and mortar combination, as shown in Table 6.6, will be the gross area stress multiplied by the ratio of the gross area divided by the net area. This analysis will therefore consider the case where the dead and live load is at this permitted limit, which could theoretically occur considering equipment for the theoretical Water Treatment Plant is to be hung from the concrete roof structure. It is worth noting that this would be an impressive load indeed to reach the permitted limit for dead and live loads. This analysis is ignoring the effects of snow, which could theoretically add additional compressive stress, because this load would be relatively minor, being limited by the short parapet.

Table 6.6 Maximum Permitted Net Area Compressive Stresses using Empirical Design for the structure analyzed in Section 6.1.3

Mortar ID	Permitted Gross Area Dead Plus Live Compressive Stress (psi)	Permitted Net Area Dead Plus Live Compressive Stress (psi)	Permitted Net Area Flexural Compressive Stress (psi)	Permitted Net Area Total Compressive Stress (psi)
CPSA-C-	112	210	114	324
CPNA-C-	97	182	114	296
CMSA-C-	112	210	114	324
CMNA-C-	97	182	114	296
CPSB-C-	79	159	122	281
CPNB-C-	73	147	122	269
CMSB-C-	79	159	122	281
CMNB-C-	73	147	122	269

Table 6.7 Measured Factors of Safety for Compression using Empirical for the structure analyzed in Section 6.1.3

Mortar ID	Control F.S.	Wet F.S.
CPSA-C-	6.2	6.2
CPNA-C-	8.0	6.0
CMSA-C-	6.3	6.3
CMNA-C-	7.0	7.6
CPSB-C-	5.9	5.3
CPNB-C-	6.2	4.7
CMSB-C-	7.0	4.8
CMNB-C-	7.0	4.7

As shown in Table 6.7, the factor of safety using Empirical for the structure analyzed is therefore calculated as the net area characteristic value for the 95% fractile compressive strengths of the masonry prisms from this research divided by the permitted net area total compressive stresses shown in Table 6.6. All factors of safety are greater than 4.7.

6.1.4 Evaluation of Mortar Cube Strengths

The minimum value of all average 28 day mortar cube strengths tested under the various simulated field conditions in this research for Type S mortars was 1351 psi, which is 75% of the 1800 psi required by ASTM C 270 Table 2 for Type S laboratory mortars.

The minimum value of all average 28 day mortar cube strengths tested under the various simulated field conditions in this research for Type N mortars was 544 psi, which is 73% of the 750 psi required by ASTM C 270 Table 2 for Type N laboratory mortars.

The minimum strength ratio of individual cube to average of three cubes was 85%.

The Property Specification of the Canadian Standard CSA A179 requires that 28 day cube strengths of laboratory mortar be at least 12.5 MPa (1813 psi) for Type S and 5MPa (725 psi) for Type N, and that 28 day cube strengths of field prepared mortar be at least 8.5 MPa (1233 psi) for Type S and 3.5MPa (508 psi) for Type N, with no individual cube strength being less than 80% of the average. This permits a field to laboratory ratio of 68% for Type S and 70% for Type N. All mortar cube strengths in this research that complied with ASTM C 270 also complied with these CSA A179 requirements. The laboratory prepared mortar prepared for –MSB--, which represents the Masonry Cement Type S “B” mortars, did not comply with ASTM C 270 because of the laboratory prepared mortar cube strength was less than 1800 psi. However, all field prepared mortar prepared for –MSB—did comply with these CSA A179 requirements.

6.2 Analysis of 28 Day Bond-Wrench Flexural Prism Results

Table 6.8 shows a summary of the characteristic 95% fractile values for 28 day prism bond-wrench flexural strengths of concrete and clay brick masonry from this research.

Table 6.8 Summary of Characteristic Values for Bond-Wrench Flexural Strength*

Mortar ID	Control Characteristic Value (psi)	Wet Characteristic Value (psi)	Wet/Control (%)	Dry Characteristic Value (psi)	Dry/Control (%)
CPSA-F-	23	39	170	24	104
CPNA-F-	21	13	62	14	67
CMSA-F-	58	31	53	0	0
CMNA-F-	31	18	58	6	19
BPSA-F-	96	77	80	62	65
BPNA-F-	74	50	68	63	85
BMSA-F-	108	50	46	44	41
BMNA-F-	110	80	73	37	34
CPSB-F-	52	20	38	25	48
CPNB-F-	0 (sic)	22	undefined	15	undefined
CMSB-F-	30	10	33	7	23
CMNB-F-	31	4	13	9	29
BPSB-F-	64	74	116	26	41
BPNB-F-	96	53	55	20	21
BMSB-F-	65	127	195	18	28
BMNB-F-	49	67	137	35	71

* Refer to Figure 1.4 for modulus of rupture values from TMS 402-08 for comparison.

The flexural strengths of masonry for this research were measured in accordance with ASTM C 1072. The following from ASTM C 1072 indicates the industry accepted application of these tests:

> "3. Significance and Use
>
> 3.1 This test method is intended to provide a simple and economical means for the determination of comparative values of flexural bond strength. It may be used either on specimens especially fabricated for bond strength evaluation or on specimens cut from existing masonry.
>
> 3.2 The bond strengths determined from this test method can be used as a means of evaluating the compatibility of mortars and masonry units. It may also be used to determine the effect on flexural bond strength of such factors as masonry unit and mortar properties, workmanship, curing conditions, coatings on masonry units, or any other factors that may be of concern.
>
> 3.3 Flexural bond strength determined by this test method should not be interpreted as the flexural bond strength of a wall constructed of the same material. However, results may be used to predict the flexural strength of a wall. Nor should it be interpreted as an indication of extent of bond for purposes of water permeance evaluation."

Large variations in results are common with bond wrench testing. These large variations are one of the reasons that bond wrench results are used primarily for comparative purposes, and not for evaluating the flexural strength of a wall. To evaluate the flexural strength of a wall, the industry considers flexural testing of full scale wall panels more appropriate.

Note that the characteristic 95% fractile value for CPNBCF- was 0 psi, which is significantly lower than any other flexural strength measured during this research. This value is not discarded because the control testing in this research represents the scatter of data for the general testing conditions of most research used by TMS 402 as the basis for determining the permitted allowable flexural tension values in TMS 402-08.

The wet limit strength or the dry limit characteristic value divided by the control condition characteristic value, expressed as a percentage in Tables 6.8, indicates the combined influence of the many conditions described in Chapter 3 to simulate the field condition limits permitted by TMS 402 and referenced standards.

For wet limit conditions, this relative strength (excluding the undefined value) was as low as 13% and as high as 195%, with an average of 83%, standard deviation of 59%, and a coefficient of variation of 71.2%.

For dry limit conditions, this relative strength was as low as 0% and as high as 104%, with an average of 46%, standard deviation of 32%, and a coefficient of variation of 70%.

Research by Thomas and Melander, both independently referenced by ASTM C 270, indicates that one may be able to reasonably predict the full-scale wall panel strengths by multiplying the values from Table 6.9 by a factor of 1.11 or 1.12 respectively (34, 35). Other research is available that indicates this factor may truly be significantly higher or lower, depending on many factors (36). However, in the absence of full-scale wall panel tests to correlate the bond-wrench strengths in this research, this analysis will predict the full-scale wall panel characteristic values by multiplying the bond-wrench values by a factor of 1.12.

For both ASD and SD, TMS 402 permits assumptions regarding flexural strengths according to tables which require knowledge of the category of cement and the mortar type, as shown in Figures 1.4 and 1.5. The same values apply to both concrete and clay masonry. To evaluate the current provisions of TMS 402, it would therefore be appropriate to combine all data related to each of the four relevant categories: Type S Portland Cement-Lime, Type N Portland Cement-Lime, Type S Masonry Cement, and Type N Masonry Cement. The data for each set of these four categories contains 36 individual bond-wrench test results, except for Type S Masonry Cement which has 18 because –MSB-- is excluded as noted below.

One should note that this flexural strength analysis will therefore combine control, wet and dry testing conditions for each of these 4 categories of mortar, so that each population is represented well by the full range of tested, permitted conditions. As defined by professional, commercial masons complying with the requirement of ASTM C 270 for the "maximum amount of water to produce a workable consistency" under different theoretical conditions requiring different levels of workable consistency: One-third of the data represents the dry limit; one-third of the data represents normal conditions; and one-third of the data represents the wet limit. One-third of the data represents 40°F curing; one-third of the data represents 75°F curing; and one-third of the data represents 90°F curing. One-third of the data represents mortar placed approximately 15 minutes after mixing without retempering; two-thirds of the data represents mortar placed approximately 120 minutes after mixing with the mason retempering the mortar one time before placement. One-half of the data represents mortar with sand meeting the gradation requirements of ASTM C 144 Sections 4.1 and 4.2; one-half of the data represents mortar with sand that did not comply with ASTM C 144 Sections 4.1 and 4.2 but complies with ASTM C 144 Section 4.4, with the exception that –MSB— laboratory prepared mortar did not comply with ASTM C 270 as noted in Section 4.2.2 of this thesis and is therefore excluded from this analysis. One-quarter of the data represents high IRA clay masonry that was not intentionally wetted; one-quarter of the data represents a low IRA clay masonry; and one-half of the data represents conventional concrete masonry. One-half of the data represents masonry with units that have been unintentionally wetted by rain; one-half of the data represents masonry with dry units. All of the data represents mortar that is batched so that it complies with ASTM C 270 by both the Proportion Method and the Property Method (noting that –MSB-- laboratory mortar did not comply with ASTMC C 270 as noted in Section 4.2.3 of this thesis and is therefore excluded from this analysis), all batches with the maximum sand and maximum hydrated lime contents by volume according to the Proportion Method. Therefore, it seems appropriate to evaluate TMS 402 according to the characteristic 95% fractile values for these populations which represent the range of permitted field conditions well.

Table 6.9 shows the characteristic 95% fractile values, based on a log normal statistical assumption, of the predicted full-scale wall panel flexural strengths (determined by multiplying each bond-wrench strength by 1.12) for the four mortar categories defined by TMS 402 Tables 2.2.3.2 and 3.1.8.2, including all of the applicable data from this research, representing a range of conditions permitted by TMS 402, TMS 602 and ASTM C 270.

Table 6.9 Characteristic Values for Predicted Unreinforced, Ungrouted Hollow Masonry Wall Panel Flexural Strengths with Tension Normal to Bed Joints

Mortar Types			
Portland Cement-Lime		Masonry Cement	
Type S	Type N	Type S	Type N
25 psi	9 psi	9 psi*	9 psi

*Note: If –MSB-- was included in the analysis, this value would be 10 psi.

6.2.1 Factors of Safety for Flexural Tension using Allowable Stress Design

For Allowable Stress Design (ASD), if the one-third stress increase is not used, the factor of safety for flexural tensile stress associated with TMS 402-08 for both unreinforced and ungrouted masonry normal to the bedjoint can be predicted to be the characteristic 95% fractile value from this research for bond-wrench flexural tensile stress at failure divided by the allowable flexural tensile stress. The allowable values for hollow, ungrouted units with flexural tensile stress normal to the bedjoint are applicable to this research. (All units in this research were less than 75% solid.) These allowable values from TMS 402-08 for both concrete and clay masonry are provided in Figure 1.5: Portland Cement-Lime Type S, 25 psi; Portland Cement-Lime Type N, 19 psi; Masonry Cement Type S, 15 psi; Masonry Cement Type N, 9 psi. If the one-third stress increase is used, the allowable values above are multiplied by 1.33 so that the factor of safety with the one-third stress increase equals the factor of safety without the one-third stress increase divided by 1.33. Refer to Tables 6.10 and 6.11 for the predicted factors of safety for flexural tensile stresses using Allowable Stress Design.

Table 6.10 Predicted Factors of Safety for Out-Of-Plane Flexural Tension using ASD without the One-Third Stress Increase*

Mortar Types			
Portland Cement-Lime		Masonry Cement	
Type S	Type N	Type S	Type N
1.0	0.5	0.6**	1.0

* Refer to Figure 1.5 for allowable flexural tensile stresses in TMS 402-08 for ASD.
**Note: If –MSB-- was included in the analysis, this value would be 0.7.

Table 6.11 Predicted Factors of Safety for Out-Of-Plane Flexural Tension using ASD with the One-Third Stress Increase*

Mortar Types			
Portland Cement-Lime		Masonry Cement	
Type S	Type N	Type S	Type N
0.8	0.4	0.5**	0.8

* Refer to Figure 1.5 for allowable flexural tensile stresses in TMS 402-08 for ASD.
**Note: If –MSB-- was included in the analysis, this value would still be 0.5.

6.2.2 Factors of Safety for Flexural Tension using Strength Design

For Strength Design (SD), the factor of safety is directly proportional to the load factor, which varies depending on the nature of the loading (dead, live, wind, etc…). There are two primary sources of flexure in masonry walls: wind and seismic. Unreinforced, ungrouted masonry is not common in high seismic areas. Furthermore, the SD load factor for seismic is unity because the probability of events is accounted for in the loading analysis, not with a load factor. In addition, the most appropriate load factor for comparison with the above Allowable Stress Design factors of safety, which were based on the assumption of an ASD "load factor" of unity, is 1.6 for wind. Therefore, this analysis considers the case of a non-load bearing wall which receives out of plane wind with a load factor of 1.6. The factor of safety is inversely proportional to the strength reduction factor, ϕ, which is 0.6 for unreinforced masonry according to TMS 402-08 Section 3.1.4. Using the TMS 402-08 SD method, therefore, the predicted wind-load-only case factor of safety for flexural tensile stress associated is the reported characteristic 95% fractile value for flexural tension stress at failure from this research multiplied by a load factor of 1.6, divided by the product of the strength reduction factor of 0.6 and f_r . The modulus of rupture values for hollow, ungrouted units with flexural tensile stress normal to the bedjoint are applicable to this research. (All units in this research were less than 75% solid.) These applicable SD modulus of rupture values, f_r, are provided in Figure 1.4: Portland Cement-Lime Type S, 63 psi; Portland Cement-Lime Type N, 48 psi; Masonry Cement Type S, 38 psi; Masonry Cement Type N, 23 psi. Refer to Table 6.12 for the predicted wind-load-only factors of safety for flexural tensile stresses using Strength Design.

Table 6.12 Predicted Factors of Safety for Out-Of-Plane Flexural Tension using SD with a Wind-Load-Only case and Unreinforced Masonry *

Mortar Types			
Portland Cement-Lime		Masonry Cement	
Type S	Type N	Type S	Type N
1.0	0.5	0.6**	1.0

* Refer to Figure 1.4 for modulus of rupture values in TMS 402-08 for SD
**Note: If –MSB-- was included in the analysis, this value would be 0.7.

6.2.3 Factors of Safety for Out-Of-Plane Flexural Tension using Empirical

Empirical Design requires that the thickness of the masonry be a minimum of 6 inches. Evaluating Empirical Design provisions requires an analysis of specific geometric conditions. Therefore this analysis will only evaluate the predicted factors of safety for the concrete masonry, which had units that were nominally 8 inches thick. This analysis is still relevant because multi-wythe clay brick masonry is generally not as common today for new construction as single wythe 8 inch concrete masonry construction.

For Empirical Design (Empirical), the predicted factor of safety for out-of-plane flexural tensile stress normal to the bedjoint in unreinforced, ungrouted masonry is the reported characteristic 95% fractile value for net area flexural tensile stress at failure from this research reported in Table 6.9 divided by the out-of-plane flexural tensile stress permitted by the maximum span to nominal thickness ratios of TMS 402-08 Table 5.5.1. The out-of-plane flexural tensile stress in a wall will vary depending on the geometric arrangement of building elements and the flexural loading under consideration. TMS 402-08 Section 5.1.2.2 does not permit the use of Empirical as part of the lateral force resisting system in Seismic Design Categories B and C, and does not permit the use of Empirical at all in Seismic Design Categories D, E and F. This analysis will consider the wind-load-only case with a single story, non-load bearing exterior wall, which may be used for comparison with the ASD and SD factors of safety provided above; the height to thickness ratio permitted for this case is 18. Because the concrete masonry units tested in this research had an 8 inch nominal thickness, this analysis will consider the maximum vertical span permitted by Empirical, which is 12 feet. At the corner of a building, the wind loads are higher; however, a return wall could provide additional lateral resistance reducing stresses. Therefore this analysis will consider the components and cladding wind load for a 12 foot long wall that is not near a corner. This analysis will also consider the critical case in which a full-story vertical opening disconnects the masonry wall on each side, because the horizontal continuity of the panel could provide flexural resistance, bridging the masonry across zones of higher loading in a wind event. Furthermore, this analysis considers the critical case in which there is a short horizontal opening (such as may occur with an 8 inch tall louver) centered on the wall as permitted by TMS 402-08 Section 5.5.1 which states, "For walls with openings that span no more than 4 feet, parallel to W_s, if W_s is no less than 4 feet, then it shall be permitted to ignore the effect of those openings." This in effect would increase the effective wind load on the wall being analyzed by 50%. (A 4 foot wide structural jamb occurs on each side of the 4 foot wide opening.) According to TMS 402-08 Table 5.1.1, Empirical is permitted for the design of exterior masonry elements of buildings that are less than 35 feet high in Basic Wind Speeds up to 110 mph. Per ASCE 7-05, the wind load required for design of buildings may be calculated by either the Analytical Method or the Simplified Method. For this analysis, the Simplified Method is used because the Analytical Method requires detailed knowledge about the structure which would make this analysis less applicable to structures across the country. According to ASCE 7-05 Figure 6-3 for 110 mph Basic Wind Speed, the critical Components and Cladding net design wind pressure using the Simplified Method for walls in Zone 4 (away from the corner), with an effective area of 144 square feet as occurs in the wall under analysis, is 20.1 psf. For Exposure Category C, this value is to be multiplied by 1.21. For an essential building, such as a Fire Station, this value is to be further increased by 1.15. Therefore, this analysis will consider the case with both conditions, so that the wind pressure for design is 28.0 psf. Given that the elevation of the opening can occur at any location along the vertical span of the wall, according to the basic structural engineering assumptions that plane strains remaining plane, using mechanics of materials analysis for the vertical span of the wall, the moment in the wall at mid-height must be at least equal to the fixed-fixed end condition moment for the uniform wind load and cannot be greater than the pinned-pinned end condition moment, with the actual stress contingent upon the actual fixity of the connections to the wall at the bottom and the top of the wall. Therefore, the average unfactored moment in a 16 in. wide strip of wall must be between 12,080 pound-in. and 18,110 pound-in.

For "A" concrete units, the average section modulus was 120.0 in.^{3}, which indicates that the average flexural stress for "A" concrete units would have to be between 101 psi and 151 psi, if the bedjoints in the wall are fully mortared as the prisms were in this research. For the fixed-fixed case, the critical axial compressive stress is 0.0 psi, occurring at the top of the wall. For the pinned-pinned case, the critical axial compressive stress is 4.8 psi, occurring at mid-height, based on the measured weights and net areas of the units, which reduces the maximum possible stress from 151 psi to 146 psi.

For “B” concrete units, the average section modulus was 119.5 in.^{3}, which indicates that the average flexural stress for “A” concrete units would have to be between 101 psi and 151 psi, if the bedjoints in the wall are fully mortared as the prisms were in this research. For the fixed-fixed case, the critical axial compressive stress is 0.0 psi, occurring at the top of the wall. For the pinned-pinned case, the critical axial compressive stress is 4.7 psi, occurring at mid-height, based on the measured weights and net areas of the units, which reduces the maximum possible stress from 151 psi to 146 psi.

Therefore, for both “A” and “B” concrete masonry units, the average out-of-plane flexural tensile stress in the wall under analysis must be between 101 psi and 146 psi.

According to the above analysis, Table 6.13 shows the predicted factors of safety for out-of-plane flexural tensile stresses using Empirical Design, calculated by dividing the reported characteristic 95% fractile value from this research for bond- wrench flexural stress at failure by the range of stresses permitted by Empirical for the condition analyzed above.

Table 6.13 Predicted Factors of Safety for Out-Of-Plane Flexural Tension using Empirical for the 12 ft. x 12 ft. exterior wall with 4 ft. wide opening described in Section 6.2.3*

Mortar Types			
Portland Cement-Lime		Masonry Cement	
Type S	Type N	Type S	Type N
0.17 to 0.25	0.6 to 0.09	0.06 to 0.09**	0.06 to 0.09

* Refer to Figure 1.6 for maximum span-to-thickness limits in TMS 402-08 for Empirical
**Note: If –MSB-- was included in the analysis, these values would be 0.07 to 0.10.

These predicted factors of safety are all less than or equal to 0.25, indicating a very unsafe condition. This significant safety issue appears to be more related to the nature of Empirical’s limitations than the properties of the masonry. For comparison, Table 6.14 presents the factors of safety for the 12 ft by 12 ft panel described above using the Empirical Design method even if one uses the unreduced modulus of rupture values from Strength Design, which in theory would model more common or “expected” field conditions as-constructed.

Note that this indicates the out-of-plane flexural tensile stress permitted by Empirical is greater than all of the TMS 402-08 Table 5.4.2 unreduced modulus of rupture values, indicating that this wall is unsafe for all mortar types if the flexural strengths are as currently recognized by TMS 402 in the Strength Design method even without reducing the strength to account for variability. Depending on which mortar type is used, the true modulus of rupture would actually need to be approximately 2 to 6 times the modulus of rupture values in the Strength Design method for the wall to be capable of withstanding the wind forces required by ASCE 7-05.

Table 6.14 Factors of Safety for Out-Of-Plane Flexural Tension using Empirical for the 12 ft. x 12 ft. exterior wall with 4 ft. wide opening described in Section 6.2.3 assuming the flexural capacity is the unreduced modulus of rupture from SD*

Mortar Types			
Portland Cement-Lime		Masonry Cement	
Type S	Type N	Type S	Type N
0.43 to 0.62	0.33 to 0.48	0.26 to 0.38	0.16 to 0.23

* Refer to Figure 1.4 for unreduced modulus of rupture values in TMS 402-08 for SD

With Type N masonry cement mortar the wall should be expected to fail bending out-of-plane at a Basic Wind Speed that is the square root of the product of 0.2 (the average F.S. in Table 6.14) multiplied by the square of 110 mph. Such a wall should be expected to fail in out-of-plane flexure at approximately 50 mph. While the case analyzed may appear to the casual reader as a “worst case” scenario, one should note that Empirical would permit this structure to be part of a prototypical design for a fire station, hospital, police station, public utility control station, or school that is repeated all along the coast of the United States in 110 mph Basic Wind Speed areas. The potential consequences to an entire community affected by a major wind event are serious indeed.

6.2.4 Factors of Safety for In-Plane Flexural Tension using Empirical

As discussed in Section 6.2.3, this analysis will only evaluate the predicted factors of safety for the concrete masonry, which had units that were nominally 8 inches thick. And, the in-plane flexural tensile stress in a wall will vary depending on the geometric arrangement of building elements and the flexural loading under consideration.

With regard to factors of safety for in-plane flexure of shear walls, this analysis will consider the Empirical Design case described in Section 6.1.3 above, which was developed for analysis of compressive factors of safety. TMS 402-08 Section 5.8.3.1 requires any "net uplift" of the roofing diaphragm (determined by an analysis of "roof loads") to be resisted entirely by an anchorage system designed in accordance with TMS 402-08 Chapter 2 for Allowable Stress Design. However, TMS 402-08 Section 5.8.3.1 does not apply to net flexural uplift at the base of shear walls due to wind pressure on exterior walls. For the case analyzed in Section 6.1.3 above, there would not be any net tension because the axial compression, which includes the enormous weight of the equipment hung from the roof in that instance, would be larger in all of the cases analyzed than the flexural tension, which is equal to the flexural compression. However, the critical condition to consider for flexural tension factors of safety is if the theoretical Water Treatment Plant did not need to hang this equipment from the roof. Therefore, this in-plane flexural tension analysis will consider the case with no such equipment. In this instance, the roof structure could consist of a 12 inch thick cast-in-place concrete roof slab. This slab has sufficient dead load to resist the uplift wind forces required by ASCE 7-05; therefore, an anchorage system is not required by TMS 402-08 Section 5.8.3.1. The dead load along the top of the shear wall from the 12 inch thick concrete roof slab is 1,500 plf. Assuming that the roof dead load would create a uniform pressure on the base of the wall, this roof dead load would create a uniform net compressive stress of 31.4 psi if 'A' units are used and 33.8 psi if 'B' units are used. The dead load of the wall itself at the bottom of the wall would create a net area compressive stress of 9.6 psi if 'A' units are used and 9.4 psi if 'B' units are used. Therefore the total dead load net area compressive stress at the base of the shear wall would be 41 psi if ‘A’ units are used and 43 psi if ‘B’ units are used. The permitted net area flexural tension stresses at the base of the shear wall are equal to the net compressive stresses shown in Table 6.6 above. Table 6.15 shows the resultant net area flexural tension stresses.

Table 6.15 Permitted Resultant Net Area Flexural Tension using Empirical Design for the 6 ft long shear wall analyzed

Concrete Masonry Product	Dead Load Net Area Compressive Stress (psi)	Wind Load Net Area Flexural Tension Stress (psi)	Resultant Net Area Flexural Tension Stress (psi)
‘A’ Units	41	114	73
‘B’ Units	43	122	79

As shown in Table 6.16, the factor of safety for in-plane flexural tension using Empirical for the shear wall case analyzed in this section is the predicted wall panel flexural tension strength shown in Table 6.9 divided by the resultant net area flexural tension stress permitted by Empirical from Table 6.15. Factors of safety less than 1.0 indicate an unsafe condition.

Table 6.16 Predicted Factors of Safety for In-Plane Flexural Tension using Empirical for the 6 ft long shear wall analyzed

Concrete Masonry Product	Mortar Types			
	Portland Cement-Lime		Masonry Cement	
	Type S	Type N	Type S	Type N
‘A’ Units	0.34	0.12	0.12*	0.12
‘B’ Units	0.32	0.11	Data Not Available**	0.11

*Note: If –MSB-- was included in the analysis, this value would be 0.14.
**Note: If –MSB-- was included in the analysis, this value would be 0.13.

The factors of safety for in-plane behavior are all less than 0.34, indicating approximately the same level of danger associated with out-of-plane behavior. Again, this appears to be more related to the nature of the Empirical provisions, which do not account for the many design variables which can occur in real structures that are not similar to the theoretical building model assumed when the Empirical provisions were developed. For example, Table 6.17 presents the factors of safety for the shear wall described in this section using the Empirical Design method even if one uses the unreduced modulus of rupture values from Strength Design, which in theory would model more common or "expected" field conditions as-constructed.

Table 6.17 Factors of Safety for In-Plane Flexural Tension using Empirical
for the 6 ft long shear wall analyzed
assuming the flexural capacity is the unreduced modulus of rupture from SD*

Concrete Masonry Product	Mortar Types			
	Portland Cement-Lime		Masonry Cement	
	Type S	Type N	Type S	Type N
'A' Units	0.86	0.66	0.52	0.32
'B' Units	0.80	0.61	0.48	0.29

Note that this indicates the flexural in-plane tensile stress permitted by Empirical is greater than all of the TMS 402-08 Table 5.4.2 unreduced modulus of rupture values, indicating that this wall is unsafe for all mortar types if the flexural strengths are as currently recognized by TMS 402 in the Strength Design method even without reducing the strength to account for variability. Depending on which mortar type is used, the true modulus of rupture would actually need to be up to approximately 4 times the modulus of rupture values in the Strength Design method for the wall to be capable of withstanding the wind forces required by ASCE 7-05.

While the shear wall case analyzed may appear to the casual reader as a “worst case” scenario, one should note carefully that Empirical would permit this structure to be part of a prototypical design for important structures that is repeated all along the coast of the United States in 110 mph Basic Wind Speed areas. The potential consequences to an entire community affected by a major wind event are yet again very serious indeed. It can be approximated that with Type N masonry cement mortar the wall should be expected to fail at a Basic Wind Speed that is the square root of the product of 0.29 (the factor of safety in Table 6.17) multiplied by the square of 110 mph. Such a wall should therefore be expected to fail in out-of-plane flexure at approximately 60 mph, which is not an uncommon wind speed for most of the United States.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions and Recommendations for TMS 402 and ASTM C 12 Committees

1. This research indicates that the current industry standard methods of specifying mortar by ASTM C 270 without a minimum required field mortar cube compressive strength, and specifying the compressive strength of masonry using the Unit Strength Method of TMS 402-08 without a minimum required field masonry prism compressive strength, are safe for conventional concrete and clay masonry even when considering the combined effect of the many extreme field conditions permitted by TMS 402-08 and referenced documents. It therefore appears acceptable, solely with regard to compressive strength, for the masonry industry to continue to not require designers to specify a minimum field mortar cube compressive strength so that masons may focus on improving other properties that are more essential to good, integrated masonry such as durable bond between units and mortar. If a minimum required field mortar cube compressive strength is desired to establish a base line for consistency, it is recommended that the required strengths be significantly lower than the minimum required laboratory mortar cube compressive strengths for the standardized laboratory mortar found in ASTM C 270 Table 2. Furthermore, it appears that it would be appropriate to modify the Commentary to TMS 602 Section 6.1, noting the following: If some assurance of a minimum mortar performance is desired, this research indicates it appears reasonable to specify that average 28 day field-prepared ASTM C 780 mortar cube strengths for Type S and Type N mortars exceed values that are approximately two-thirds of the ASTM C 270 Table 2 values (e.g. 1200 psi for Type S field-prepared

mortar and 500 psi for Type N field-prepared mortar), with no individual cube strength being less than 80% of the average of three cubes, which would be consistent with the Property Specifications of the Canadian Standard CSA A179 that have been used successfully for several decades.

2. This research indicates that the combined effect of the many extreme field conditions permitted by TMS 402-08 and referenced documents can significantly reduce the flexural bond strength of masonry compared with the testing conditions which were the primary basis used historically in establishing the permitted flexural strength assumptions that are currently in TMS 402-08. The characteristic 95% fractile values for flexural strength under the "dry limit" set of conditions defined in this research were on average 46% of the characteristic values for the control conditions; under "wet limit", 83%. The reported factors of safety for ASD without the one-third stress are the same as for SD because the SD values were established by multiplying the ASD values by 2.5 which is very similar to the load factor for wind of 1.6 divided by the applicable strength reduction factor of 0.6. These reported factors of safety ranged from 0.5 to 1.0, indicating it would be appropriate for TMS 402 to consider reducing the permitted flexural tension strengths. The reported factors of safety for ASD with the one-third stress increase ranged from 0.4 to 0.8, indicating a safety concern for all categories. Therefore, this research indicates the one-third stress increase does not appear to be safe. This further indicates that the increase to allowable flexural tension stresses for unreinforced, ungrouted masonry in the TMS 402-committee-approved working draft of revisions to TMS 402-08 dated November 15, 2009 appears to yield unsafe results and results that will not be consistent with SD. It is therefore recommended that the increase to these values shown in the TMS 402-committee-approved working draft of revisions to TMS 402-08 dated November 15, 2009 be repealed, considering that the

reliability study which formed a significant basis for this increase was based on testing under relatively standardized conditions and not the combined effect of the many extreme field conditions permitted by TMS 402-08 and referenced documents. The predicted factors of safety were obtained by increasing the prism strengths to obtain wall panel strengths. Some may believe that the increase was not sufficient because a larger panel of masonry would have more redundancy. However, one should note that a masonry wall is defined by TMS 402 as an element with a horizontal length that is at least three times its thickness, which would be only 50% longer than the concrete masonry prisms tested in this research for the critical case. It is recommended that, at a minimum, the proposed increase in allowable stress for ungrouted, unreinforced masonry be repealed until a time when full-scale testing of flexural wall panels under similar conditions is shown to justify the increase. Note that this research does not address partially or fully grouted masonry values; therefore, this research does not indicate that the proposed increase for partially or fully grouted masonry should be repealed.

3. This research indicates that the Empirical Design provisions of TMS 402 are fundamentally flawed. In the past, many others have questioned the suitability of the Empirical provisions based on a comparison of results with ASD and SD. This thesis comes to this conclusion by a combination of strength measurement and analysis of essential facilities that would be permitted by TMS 402-08. The factors of safety associated with the Empirical Design provisions of TMS 402-08 are extremely low for the extreme cases analyzed in this thesis. A factor of safety less than 1.0 indicates an unsafe condition. Depending on the type of mortar used, the predicted factors of safety for the extreme cases analyzed in this thesis ranged from 0.06 to 0.25 for the out-of-plane flexure and from 0.11 to 0.34 for in-plane flexure. Furthermore, the analyses of

extreme cases in this thesis show that even if the modulus of rupture values from the Strength Design method of TMS 402-08 are used without any strength reduction factor, the factors of safety range from 0.16 to 0.62 for out-of-plane flexure and from 0.29 to 0.86 for in-plane flexure. The International Existing Building Code, IEBC, defines any portion of a building as “dangerous” if it is not capable of withstanding a wind pressure of two-thirds that required by the IBC for new structures without exceeding the normal strength permitted by the IBC for such structures (38). The Empirical provisions permit structures to be constructed which the IEBC would define as dangerous using ASD or SD strength values the day they are built. Some may refer to the historical use of Empirical and consider the structures and conditions analyzed in this research as atypical; however, the essential facilities analyzed are nonetheless permitted and could be prototypical structures repeated for specific applications across the United States. Some may simply not believe the results of the analyses in this thesis because they may not be aware that unreinforced structures do fail in flexure around the country. Anecdotal evidence of failures is often illogically dismissed by anecdotal evidence of surviving structures. The levees of New Orleans worked for many years until they failed during Hurricane Katrina; anecdotal evidence is not a sufficient method of determining reliability for essential facilities. Considering the potential impact of failure, it is not logical to permit essential facilities such as those analyzed in this thesis to be designed using the Empirical provisions of TMS 402. However, the TMS 402 Committee recently failed to pass a ballot attempting to prohibit the use of the Empirical method for essential facilities. There are many reasons some structures have survived, for example shielding of wind by adjacent structures, or sharing of lateral force resisting systems by adjacent structures. However, the purpose of the building code is not to permit construction which might possibly survive; the purpose of the building code is to prohibit practices which are not sufficiently reliable. Even if the Empirical Design provisions are modified in future editions of TMS 402 to prevent the specific cases analyzed in this thesis, one can use the analyses in this thesis to quickly show that there are still legitimate life safety concerns at many less critical cases including but not limited to: slower basic wind speeds, walls without openings, shear walls with flanged components, smaller length-to-width roof diaphragm aspect ratios, and smaller height-to-length shear wall aspect ratios. The Empirical provisions only apply to unreinforced masonry, which is becoming less common in the United States market with time, and Empirical does not apply to many geographical areas of the United States. On the other hand, TMS 403-10 “Direct Design Handbook for Masonry Structures” now provides an alternative to Empirical, fully complying with the TMS 402-08 Strength Design provisions, providing design options for both reinforced and unreinforced (both ungrouted and fully grouted) concrete masonry, being easier and faster to use than both ASD and SD methods of TMS 402 for many common, simple structures, and applying to the vast majority of the United States (37). TMS 403-10 was developed by the TMS Design Practices Committee at the request of the TMS 402 Veneer, Glass Block, and Empirical Subcommittee to develop a method that is easy to use and consistent with the results of ASD and SD so as to replace Empirical. It is recommended that TMS 402 immediately address the life-safety concerns identified by the tests and analyses of this research. There are many ways TMS 402 could address these life-safety concerns. The many simplifying assumptions made during the historical development of the Empirical Design provisions were clearly not all incorporated into the requirements of TMS 402 Chapter 5 for Empirical Design. It would be a major undertaking indeed to attempt to define these assumptions in a modern context as has been done for TMS 403-10. Therefore, the masonry industry should consider removing the Empirical Design Chapter (Chapter 5) from future editions of TMS 402. In the near future, however, the TMS 402 Committee may wish to implement a limited approach that simply prohibits the use of hollow, ungrouted masonry and prohibits design of essential facilities by the Empirical Chapter, to address the life-safety issues identified in this research and study further the safety of the remaining types of masonry permitted, perhaps moving the Empirical Chapter to an Appendix of TMS 402. This limited approach may be reasonable for out-of-plane bending considering that all nonbearing masonry walls have the same height to thickness limit even though solid or fully grouted masonry has a significantly greater weak axis section modulus than hollow, ungrouted masonry. In a similar manner, this limited approach may be reasonable for in-plane bending considering that the same geometric limits regarding shear walls apply to both solid or fully grouted masonry as well as hollow, ungrouted masonry walls even though the solid or fully grouted masonry has a significantly greater strong axis section modulus than hollow, ungrouted masonry. Furthermore, this limited approach would make the Empirical Chapter more consistent with the historical heritage of the provisions and more consistent with the gross area stress analysis required by Empirical. It is therefore recommended for the near future that TMS 402 modify the Empirical Chapter by prohibiting design of essential facilities according to Empirical provisions, by adding a requirement that all masonry be constructed with solid units or be fully grouted, by deleting the allowable gross area compressive stresses for hollow masonry, by deleting the height to thickness limit for bearing walls that are "other than solid units or fully grouted", and by moving Chapter 5 for Empirical to a mandatory Appendix of TMS 402 until the safety of the method, after such modification, has been further evaluated.

4. This research indicates, in concert with a great deal of other research, that permitting flexural tensile strengths based only on knowledge of the unit material, mortar type and type mortar should be used with very low values indeed. However, unreinforced

masonry when designed properly can provide a superb construction product that will outlast many if not all other types of construction. The strength of the bond can even exceed the flexural capacity of the unit in some cases. Considering that much research is available indicating that flexural tensile strength depends on many project-specific factors (refer to Appendix A), it is recommended that TMS 402 develop provisions which would permit the use of bond-wrench data with a statistically significant number of specimens in establishing a much higher modulus of rupture for a defined set of conditions limiting the many parameters discussed in this thesis and in Appendix A. This "bond-wrench strength method" would be analogous to the compressive "prism strength method" so that these more practical flexural values could be used when unreinforced masonry is desired on a project. It is even possible that such a method could be used in areas where unreinforced masonry may be prevalent to permit higher flexural strength values for trained masons with conventional materials so that unreinforced masonry can be designed more economically in these areas. Therefore, it is recommended that development of a "bond-wrench strength method" be considered if there is a demand in the construction market.

7.2 Recommendations for Additional Investigation

1. This test data and other research could be used to further evaluate the effect of the strength reduction factor in Strength Design separately from the nominal strength, with regard to both flexural and compressive strengths. For example, this research seems to indicate that a lower strength reduction factor may be more appropriate with a higher compressive strength of masonry. The product of these two values is relevant in SD to the structural capacity of the masonry system; however, it may be appropriate to recognize a higher compressive strength of masonry and accept a lower strength reduction factor considering that the modulus of elasticity according to TMS 402 is a

function of the compressive strength of the masonry. The modulus of elasticity for masonry relative to that assumed for steel is relevant because this value has a large effect on the maximum permitted reinforcement ratio, which limits the amount of reinforcement a designer may install in the wall so as to ensure that the failure mode is ductile and not brittle. If an increase in this reinforcement ratio is appropriate, this would make masonry more economical in many instances.

2. This test data could be used to evaluate different proposals for minimum field cube strength specifications for quality control. Additional research could also be performed as an extension of this research. Standardized test field mortars could be developed at a given target compressive strength (which this research indicates should be significantly below ASTM C 270 Table 2 values) for the purpose of evaluating masonry prism strengths with the standardized test field mortar, as well as durability and water permeance. With this additional research, perhaps a consensus standard in the United States could be established for field cube strengths.

3. This test data could be used in conjunction with additional research to investigate the combined effect of the test conditions of this research on larger specimens such as wall panels. Such research would helpful in better evaluating appropriate flexural tension limits for TMS 402.

4. This test data could be used in conjunction with additional research to investigate the effects of temperature ranges beyond the limits tested in this research. Permitted temperature ranges beyond the limits of this research are common, yet they require additional procedures that create more variability to test results. For example, it would be appropriate to test the flexural capacity of walls with significantly higher

temperatures using various methods of protection considered appropriate by the industry.

5. Additional research could be performed to survey designers and masons across the United States and attempt to determine how much and in what areas masonry construction is unreinforced and ungrouted, and how much of that construction is designed by ASD, by SD and by Empirical.

6. This test data could be used in conjunction with additional research to explore the suitability of cone penetrometer readings in effectively establishing lower and upper limits on mortar water content in the field so as to avoid overly-dry mortar. This may be appropriate considering the subjective nature of the requirement in ASTM C 270 Section 7.3 for "a maximum amount of water to produce a workable consistency" in which workable consistency will depend on the mason, the climate, and the nature of the work being performed. Such a study could also attempt to effectively determine how dry and how wet mortar is typically laid in the field now.

APPENDIX A

“FACTORS AFFECTING BOND STRENGTH OF MASONRY”
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MASONRY

INFORMATION

Factors Affecting Bond Strength of Masonry

INTRODUCTION

The development of bond between masonry mortar and masonry units is significant to the performance of the masonry in service. Intimate contact between mortar and unit aids in resistance of water penetration. In unreinforced masonry designed by working stress analysis, the adhesion of mortar to units is relied upon to resist flexural tension stresses resulting from eccentric axial loads, or out-of plane loads, or both. These are two distinctly separate aspects of bond. The first relates to the extent of bond or degree of contact between mortar and unit. The second is the bond strength of masonry. Both are functions of several variables associated with the specific mortar and units considered, as well as the conditions under which they were assembled and cured.

Research has shown that the extent of bond and bond strength do not necessarily relate to one another. It is quite possible to develop excellent and intimate contact between mortar and unit using combinations of materials that yield relatively low bond strengths. Conversely, a high bond strength does not necessarily mean that complete and intimate contact between mortar and unit has been achieved.

Several standard test procedures have been established to measure the bond strength between mortar and unit (see PCA publication IS277, *Bond Strength Testing of Masonry*). Although the precision of these methods has not been determined and relationships between test results and in-place performance are still being investigated, there is a fairly large body of published research available from which to evaluate the test procedures and gather information on factors affecting bond strength. This document summarizes findings of research investigating the relationship of bond strength to material properties, fabrication procedures, and curing conditions.

EFFECT OF SPECIFIC VARIABLES

Variables affecting the bond strength include:

- Mortar properties
- Type of masonry unit
- Techniques used to fabricate masonry assemblies
- Specimen conditioning between fabrication and testing
- Testing procedures

Mortar Properties

ASTM C270, the Standard Specification for Mortar for Unit Masonry, classifies mortars as Types M, S, N, or O. These mortars may be specified either under the property specifications or the proportion specifications of that standard. Under the property specifications, laboratory prepared and tested mortars of each designation by type must meet specific requirements for compressive strength, water retention, and air content. The proportion specifications define a range of acceptable proportions for mortar ingredients for each mortar type. Requirements are given for both cement-lime mortars and masonry cement mortars under each set of specifications as indicated in Tables 1 and 2. Mortar properties which influence bond strength development include cement content, water retentivity, air content, and flowability as related to water content.

Cement content. Increased portland cement content of mortar generally provides increased bond strengths. Melander and Conway [Ref. 1] documented this relationship for tests conducted on concrete masonry brick and portland cement-lime mortar assemblies. They developed a mathematical model for the relationship under laboratory test conditions. Wright [Ref. 2] also observed a correlation between increased cement content of mortar and higher bond strengths

Table 1. Proportion Sprcification Requirements*

Mortar	Type	Portland Cement or Blended Cement	Masonry Cement M	Masonry Cement S	Masonry Cement N	Hydrated Lime	Aggregate Ratio
Cement-lime	M	1	–	–	–	1/4	Not less than 2 1/4 and not more than 3 times the sum of the volumes of cement and lime used.
	S	1	–	–	–	over 1/4 to 1/2	
	N	1	–	–	–	over 1/2 to 1 1/4	
	O	1	–	–	–	over 1 1/4 to 2 1/2	
Masonry cement	M	1	–	–	1	–	
	M	–	1	–	–	–	
	S	1/2	–	–	1	–	
	S	–	–	1	–	–	
	N	–	–	–	1	–	
	O	–	–	–	1	–	

* Adapted from ASTM C270.

Table 2. Property Specification Requirements*

Mortar	Type	Minimum 28-Day Compressive Strength, psi (MPa)	Minimum Water Retention, %	Maximum Air Content, %	Aggregate Ratio
Cement-lime	M	2500 (17.2)	75	12	Not less than 2 1/4 and not more than 3 times the sum of the volumes of cement and lime used.
	S	1800 (12.4)	75	12	
	N	750 (5.2)	75	14**	
	O	350 (2.4)	75	14**	
Masonry cement	M	2500 (17.2)	75	_**	
	S	1800 (12.4)	75	_**	
	N	750 (5.2)	75	_**	
	O	350 (2.4)	75	_**	

* Adapted from ASTM C270.

** When structural reinforcement is incorporated in the cement-lime or masonry cement mortar, the maximum air content shall be 12% or 18% respectively.

for masonry specimens constructed using concrete masonry brick. Their study included laboratory formulated masonry cement mortars and portland cement-lime mortars.

Some studies [Ref. 3] involving masonry assemblies constructed of cement-lime mortars and clay masonry unis have reported optimum bond strengths using cement contents in the range of Type S proportions with little or no increase when Type M proportions were used. That fact is probably related to the relative workability and water retentivity of the two types of mortars, coupled with the testing and curing conditions under which the experimental programs were conducted. Current design criteria adopted by model codes and design standards in the United States are consistent with the finding that little or no increase in bond strength is achieved by the use of Type M mortars as compared to Type S mortars. These documents (Uniform Building Code and ACI 530/ASCE 5/ TMS 402, which is adopted by reference in the BOCA Code and Standard Building Code) assign the same allowable flexural tensile stress values for masonry constructed using Type M mortar as for Type S mortar. (Allowable flexural tensile stress values listed in the codes and standards are used in the design of unreinforced masonry in which the flexural tensile resistance of the masonry is taken into consideration.) Reduced allowable stress values are listed for Type N mortars since lower bond strengths are typically obtained with Type N mortar than with Type S mortar due to the reduction in portland cement content.

Some tendency for bond strengths to increase as compressive strengths of mortars increase was observed by Fishburn [Ref. 4]. However, subsequent studies have reported little or no correlation between the compressive strength of mortar and bond strength [Ref. 5 & 6]. Since compressive strengths are influenced by cement content, one would expect bond strengths to increase with increasing cement content. However, other factors that increase bond strength, such as increased water content, tend to reduce the compressive strength of mortar. That fact may explain the lack of correlation between bond strength and compressive strength in many studies.

Water retentivity. Conflicting findings on the relationship of water retentivity to bond strength have been reported. Ritchie and Davison [Ref. 7] indicated that improved bond strengths were achieved with mortars having higher water retentivity – particularly when used with high-absorption units. However, other studies have reported either no significant improvement in bond strength with increased water retentivity [Ref. 5] or even a reduction in bond strength for mortars having higher water retention values [Ref. 8]. Apparently, specific study conditions greatly influence test results. In particular, limitations of the water retention test, procedures used to control water retention values for test mortars, and different curing and fabrication procedures have probably contributed to divergent observations. *The most convincing statement with respect to water retention of mortar is that a compatible balance with unit properties is needed. That is, fluid paste from the mortar needs to readily flow into the surface irregularities of the masonry unit, while sufficient water for cement hydration must be retained in the mortar system.* It should also be noted that the standard ASTM test used to measure water retention actually measures the ability of a mortar to retain its flow under suction rather than the ability of mortar to retain its mixing water.

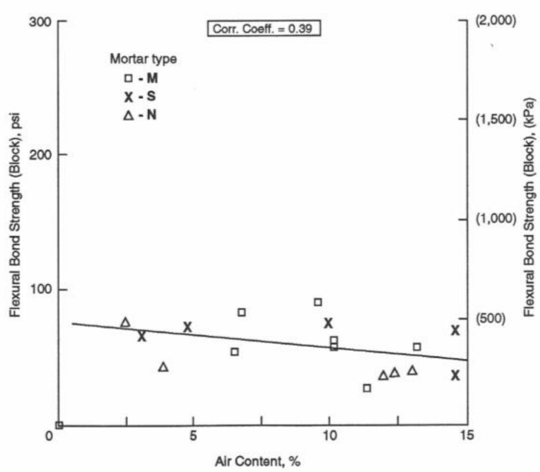

Fig. 1 — Ribar reported a linear relationship between mortar air content and bond strength. The low correlation coefficient indicates that other variables also significantly affect bond strength [Ref. 9].

Air Content. Air content of masonry mortars generated during mixing is influenced by the cementitious materials, sand, mixing procedures, water content, ambient temperature, and air-entraining admixtures. As measured by ASTM test methods on the fresh mortar, air content includes entrained air and entrapped air. Petrographic examination of the hardened mortar can be used to distinguish entrained air from entrapped air based on the air void size, shape, and spacing. Entrained air within the masonry mortar improves workability and durability of the mortar, while serving as a water reducing agent.

As air content of mortar is increased, bond strength tends to decrease provided other factors are held constant. The relationship is generally linear, as indicated in Fig. 1. The data does not support the frequently stated contention that air content levels below 10 or 12 percent do not affect bond strength; nor is placing a limit on air content of mortar an effective means of predicting or controlling the bond strength of masonry. Excellent bond strengths can be achieved using air-entrained mortars. Air content is simply one of several variables that affect bond strength. Its relative significance depends on the degree of control exercised on other variables.

Illustrating that principle, experimental work by Wright [Ref. 2] indicated that under controlled laboratory conditions of mortar mixing, specimen fabrication, and curing, the percentage of portland cement contained in the mortar had four times as much impact as the percentage of air on bond strength. Robinson and Brown [Ref. 5] reported bond strengths ranging from 30 to 179 psi [200 to 1230 kPa] for masonry assemblies fabricated with mortars having essentially equivalent air contents (18 to 19 percent). These air contents were obtained using different air-entraining agents or different procedures. They postulate from this data that, in addition to air content level, the type of air-entraining agent and bubble structure influence bond strength. Interestingly, Kampf [Ref. 10] observed improved bond strengths with increasing air contents of mortars when used with a high suction (IRA) brick, but noted a reduction in bond strengths with increasing air content of mortars when used with low suction brick. He concluded that with high suction units, mortar characteristics of workability and water retention are more significant to bond strength development, while the cohesive or tensile strength of the mortar tends to dominate bond strength development with low suction units.

Water Content. Past research [Ref. 5, 6, & 7] indicates that bond strength is significantly influenced by the water content of the masonry mortar at the time of specimen fabrication. Mortars with increased water contents produced higher bond strengths. This finding indicates that a more fluid mortar "wets out" the interface between mortar and unit better. Specification ASTM C 270 is consistent with this finding since C 270 stipulates mixing mortar "...with the maximum amount of water to produce a workable consistency." Assuming that evaluation of workable consistency includes consideration of unit characteristics, water additions to masonry mortar are self regulating. That is, too low water contents affect workability and too high water contents affect joint thickness as well as workability. Although laboratory compressive strength of mortar decreases with increased water content, bond strength increases, as shown in Fig. 2.

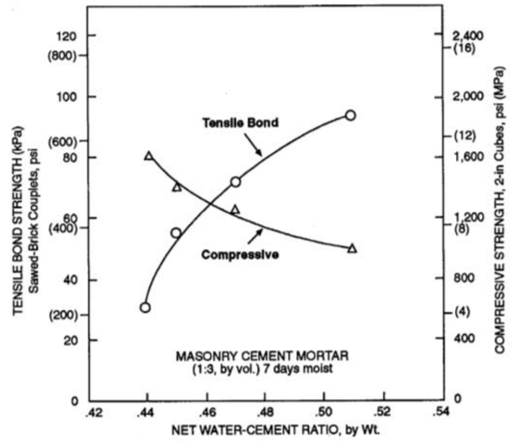

Fig. 2 — Increased mortar water content reduces compressive strength but increases bond strength [Ref. 11].

This finding also supports the practice of retempering mortars as frequently as necessary provided the mortar has not stiffened due to cement hydration. Specification ASTM C270 stipulates that mortar be used within 2 1/2 hours from the time of initial mixing to preclude the use of mortar that has hardened from hydration of the cement.

Masonry Units

Bond strength is significantly influenced by the composition and physical characteristics of the masonry units – particularly surface characteristics. Clearly, loose sand particles, dirt, or other contaminants on the surface of units will reduce the ability of the mortar to adhere to the unit. Surface texture of units also affects bond strength. A smooth, even surface limits mechanical keying with mortar, resulting in lower bond strengths than would be achieved with a unit having a rough textured surface.

For clay masonry, initial rate of absorption, later age absorption characteristics, and bedded surface texture of the unit are important. The initial rate of absorption (IRA) of the unit attracts the mortar to its surface. Once bedded and during mortar setting and hardening, mortar must retain sufficient water to promote cement hydration and crystal growth at the brick-to-unit-interface. Both surface texture and mortar composition influence this occurrence. Additionally, water absorbed by the unit is available later for continued hydration of the cement in the masonry mortar.

Several studies have documented a relationship between IRA and bond strength. Palmer [Ref. 12] observed that optimum bond strengths were obtained using units having moderate suction, (IRA in the range of about 20 g per 30 sq in. (194 sq cm) per minute), as shown in Fig. 3. Subsequent data reported by Ritchie [Ref. 7], Dubovoy [Ref. 6], and McGinley [Ref. 13] support the theory that optimum bond strengths are obtained using units having moderate suction. Kampf's work included wire cut and molded brick having IRAs ranging from approximately 6 to 75 g per 30 sq in. (194 sq cm) per minute. He reported that bond strengths tended to be lower as IRAs increased, as shown in Fig. 4. Bond strengths developed using units having similar IRAs were lower for molded brick than for wire cut brick, reflecting the influence of surface texture.

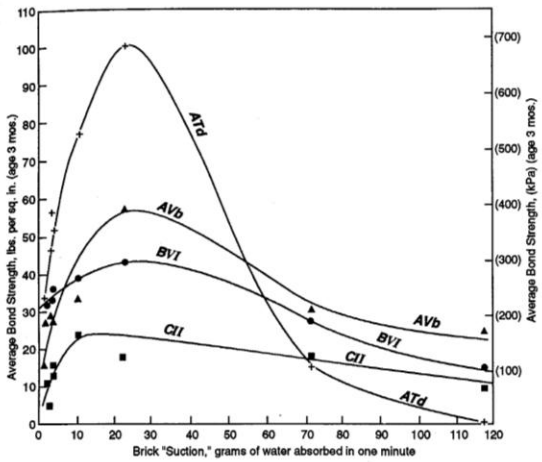

Fig. 3 — Palmer observed optimum bond strengths for brick having moderate initial rates of absorption (IRA) [Ref. 12].

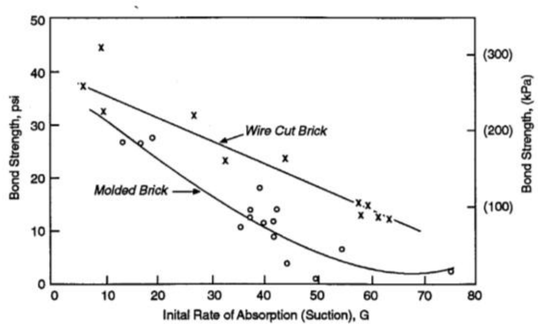

Fig. 4 — Kampf observed a trend of lower bond strengths with high IRA units and noted a difference between bond strengths achieved using wire-cut brick as compared to molded brick [Ref. 10].

Other research reports have questioned the significance of IRA with respect to bond strength development [Ref. 5 & 14], and it should be noted that IRA is a single variable associated with a component material of the masonry assembly. Unit characteristics such as surface texture and pore structure also affect bond strength and can be more significant than IRA. Dubovoy and Ribar [Ref. 6] developed a procedure for quantifying the surface texture of units in a single parameter called the contour ratio. They developed a model equation for the bond strength of masonry that included the contour ratio and the square of the IRA, as shown in Fig. 5. Predicted bond strength values show fairly good agreement with actual measured values and demonstrate that properties of clay masonry units affect bond strength at least as much as properties of masonry mortars.

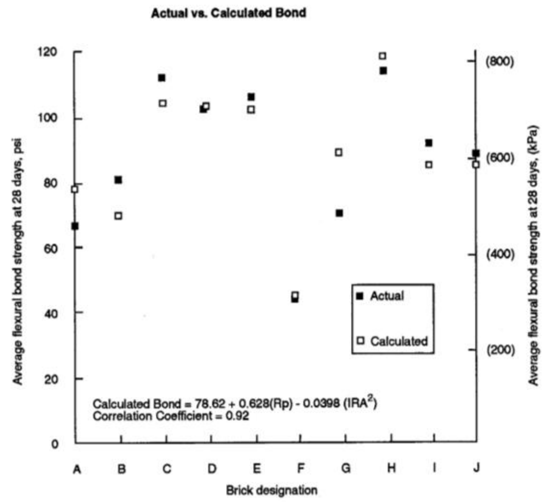

Fig. 5 — Dubovoy and Ribar developed a model equation for bond strength using brick texture (Rp) and the square of the initial rate of absorption (IRA^2) as independent variables [Ref. 6].

Research by Ritchie and Davison [Ref. 7] indicated that improved bond strengths are achieved by wetting high IRA brick. This finding is consistent with the advisory note contained in ASTM C216 which recommends that clay masonry units having an IRA over 30 g/ 30 sq in. per minute be dampened to reduce the effective IRA to a value below 30 g/ 30 sq in. per minute.

Some studies [Ref. 15] have indicated that higher bond strengths are achieved with clay masonry units than with concrete masonry units. However, published data involving bond tests of these two materials under comparable conditions of testing are very limited. Recent research conducted by the National Concrete Masonry Association indicates that very high bond strengths may be achieved in concrete masonry assemblies when Type M or S mortars are used and when the assemblies are adequately cured. Bond strengths between high strength mortars and concrete masonry units can in fact exceed the tensile strength of the concrete masonry units themselves under optimum curing conditions.

Specimen Fabrication

The preparation of test specimens for measuring bond strength varies. This preparation may involve detailed laboratory procedures or simply "qualified masons" using their own procedures on the project. Laboratory procedures concentrate on alignment, mortar bed thickness, pressure applied during setting of upper units, and tooling of joints. Specimens fabricated at the jobsite involve different craftsmen performing the above mentioned tasks without benefit of frames and guides and under varied climatic conditions.

Bond strength is highly dependent on the techniques used to bring mortar and unit together to form a masonry assembly. Important variables are: the elapsed time between applying bedding mortar on a unit and placement of the unit covering that mortar joint, the compaction of the mortar during placement of the unit, and subsequent disturbance of a unit after initial placement.

Elapsed time. Bond strength decreases as elapsed time between the application of bedding mortar to a unit and placement of the next unit increases [Ref. 7, 10]. The explanation for this is related to the previously noted relationship between bond strength and mortar water content. Mortar that is placed in contact with an absorptive masonry unit mortar immediately begins to lose moisture to that unit. Thus, the mortar will have a lower water content and a stiffer consistency when the upper unit is placed, reducing the bond between mortar and unit. Absorption characteristics of the unit, water retentive properties of the mortar, and ambient conditions also influence the significance of elapsed time on bond strength.

Compaction of mortar. As a unit is placed, it is pushed into the existing mortar bed, extruding excess mortar from the joints. This physical action forces mortar into surface irregularities of the unit. Richie and Davison [Ref. 7] compared fabrication procedures which included use of a "2-pound drop-hammer," a "4-pound drop-hammer," and a "bricklayer." They observed that bond strengths were greater for specimens fabricated using the heavier drop-hammer. The best results were achieved by the mason using normal bricklaying techniques. Some laboratory test procedures incorporate the use of drop-hammers in the fabrication of specimens to control the variable of workmanship and eliminate the need to employ a trained mason to fabricate specimens. It should be recognized that these procedures are primarily for comparative evaluation of materials. Bond values obtained in the laboratory are likely to differ from those achieved under field fabrication conditions.

Realigning units. Retaining undisturbed contact between mortar and unit immediately after placement is required to maintain optimum development of bond strength. If mortar has stiffened prior to final alignment, complete loss of bond is likely. Kampf [Ref. 10] studied the effect of disturbing the top unit of crossed-brick couplet specimens at various time intervals after placement. He observed that the elapsed time during which brick could be realigned without destroying the bond is greatest for low-suction brick and high water-retentive mortars. For one specimen, bond was destroyed by movement 15 seconds after placement. This reinforces the recommendation that once a unit is placed, alignment should be immediate. Also, any forces exerted to achieve alignment should be toward the existing mortar bed.

Effect of Curing Conditions

Conditioning of the test specimen during the period from fabrication until testing significantly influences the measured bond strength. Because portland cement paste requires a moist environment to maintain its relative humidity (above about 80%) for continued cement hydration, any condition promoting drying of the masonry mortar affects bond strength. Additionally, any drying of a masonry assembly produces restrained and differential shrinkage of the masonry mortar within the assembly. These shrinkage stresses, unless equilibrated, affect the measured bond strength. The most significant portion of the mortar prone to this effect is the mortar at the joint surface, the same mortar surface that receives the greatest tensile stress during flexural testing.

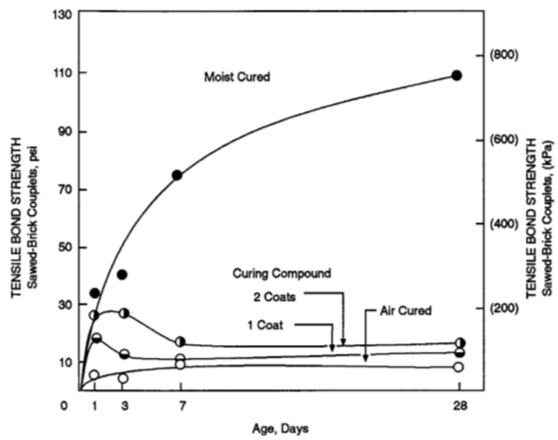

Fig. 6 — Curing conditions affect bond strength results [Ref. 11].

Ideally, with the objective of establishing the bond strength of in-place masonry, all masonry test assemblages should be subject to the same curing conditions as the actual walls they represent. However, to remove climatic conditions as an uncontrolled variable, laboratory fabrication of specimens is completed in a controlled environment. ASTM C1072, the Standard Method for Measurement of Masonry Flexural Bond Strength, recommends that "... laboratory air be maintained at a temperature of 75 ± 15°F (24 ± 8°C), with a relative humidity between 30 and 70 percent." These conditions do not constitute an effective curing environment, so some test methods require "self curing," obtained by enveloping test specimens within plastic bags. Recent comparisons of bond strengths of damp cured concrete masonry specimens as compared to specimens cured in laboratory air indicate that over a 300% increase in bond strength is achieved by damp curing.

As noted in Fig. 6, bond strength is affected by the period of moist curing and storage in air. To establish the bond strength potential of a combination of mortar and masonry units, the test specimens should be cured in a moist environment until tested.

Effect of Test Method on Bond Strength

Different test procedures produce different measured bond strength results. For a more detailed discussion of the variables involved in testing and the effect of various standard test methods, see PCA Publication *IS277, Bond Strength Testing of Masonry.*

SUMMARY

Development of bond strength in masonry is a complex process dependent on many variables related to materials, fabrication, curing, and testing. Several of these parameters are interdependent. Certain general relationships have been established by isolating and measuring the effect of specific variables. However, it is important to note that relationships exhibited under controlled experimental conditions may be obscured in actual application by the effect of changes in other parameters. Perhaps the most significant finding that can be gleaned from a review of the numerous investigations with respect to bond strength is the observation that it is a combined property of the mortar and the unit together. It cannot be accurately predicted from individual characteristics of the component materials.

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CAUTION: Contact with wet (unhardened) concrete, mortar, cement, or cement mixtures can cause SKIN IRRITATION, SEVERE CHEMICAL BURNS, or SERIOUS EYE DAMAGE. Wear waterproof gloves, a long-sleeved shirt, full-length trousers, and proper eye protection when working with these materials. If you have to stand in wet concrete, use waterproof boots that are high enough to keep concrete from flowing into them. Wash wet concrete, mortar, cement, or cement mixtures from your skin immediately after contact. Indirect contact through clothing can be as serious as direct contact, so promptly rinse out wet concrete, mortar, cement, or cement mixtures from clothing. Seek immediate medical attention if you have persistent or severe discomfort.

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An organization of cement manufacturers to improve and extend the uses of portland cement and concrete through market development, engineering, research, education, and public affairs work.

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END OF APPENDIX A

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BIOGRAPHICAL INFORMATION

Benchmark H. Harris, P.E., S.E., M.S., SECB, LEED AP BD+C received his Bachelors of Science in Civil Engineering from the University of Texas at Austin in May of 1997. In Austin, Ben worked for the Construction Materials Research Group at the J.J. Pickle Research Center for several years, obtaining Level I and Level II Certification as an ACI Laboratory Technician.

He has 13 years of experience as a Structural Engineer in North Texas and is an Associate with CHA, Inc, a national multi-discipline Engineering firm. While in North Texas, Ben obtained NCMA Testing Technician Certification and has been involved with training and quality assurance of masonry testing for the Construction Materials Testing Division of CHA. He conducted independent research at this laboratory investigating the compressive strength of masonry with improperly batched mortar, which was presented at the 10th North American Masonry Conference and which served as the foundation for some of this research.

He has been active in The Masonry Society for the past 7 years and received the TMS Service Award in 2008. He is the current Chair of the TMS Design Practices Committee and the current Program Leader for the TMS Disaster Investigation Program. He is also currently a Voting Member of TMS 402, also known as the MSJC, and has contributed to the 5th and 6th editions of the *Masonry Designer's Guide*.

He is interested in streamlining conventional masonry design and was the Chair of the group that initially developed TMS 403 *Direct Design Handbook for Masonry Structures*. In the future, he hopes to work with others to standardize an economical method of fixing a common design weakness in historic mercantile unreinforced brick buildings, which can be found extensively throughout the United States, by improving the roof/floor diaphragm-to-wall connections and roof uplift anchorage, for both life safety protection and historic preservation.